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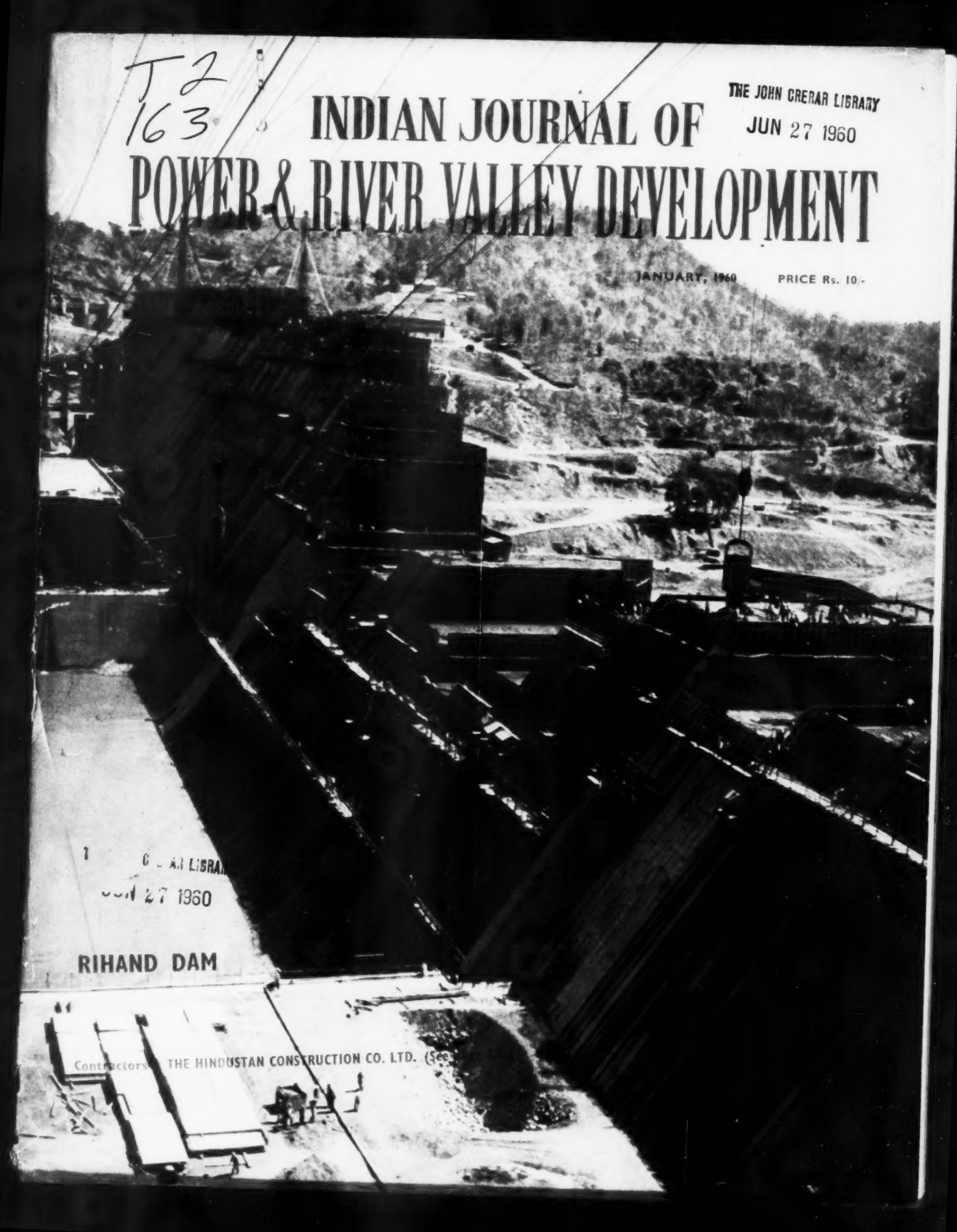

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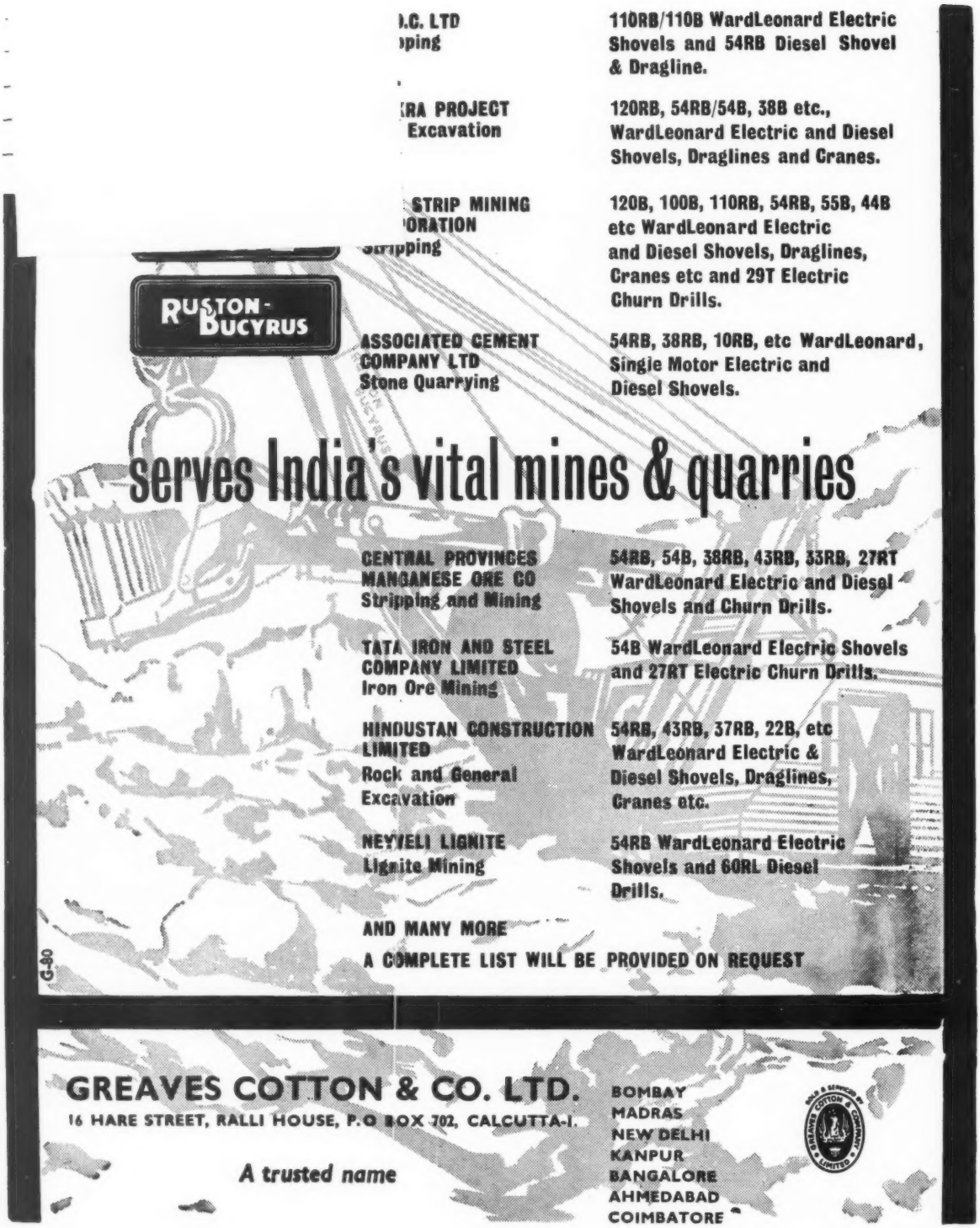

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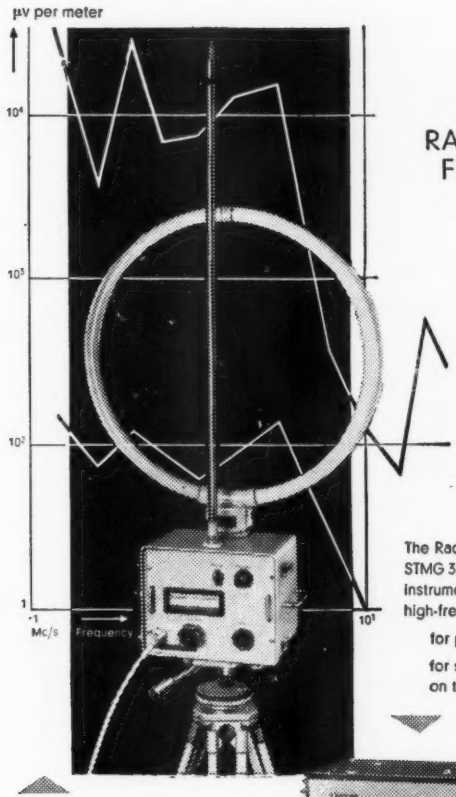

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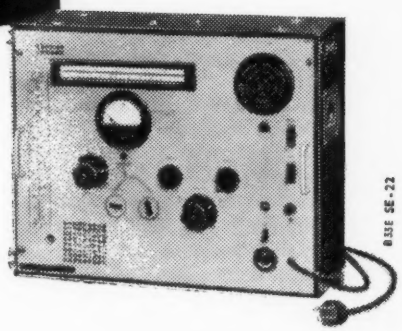

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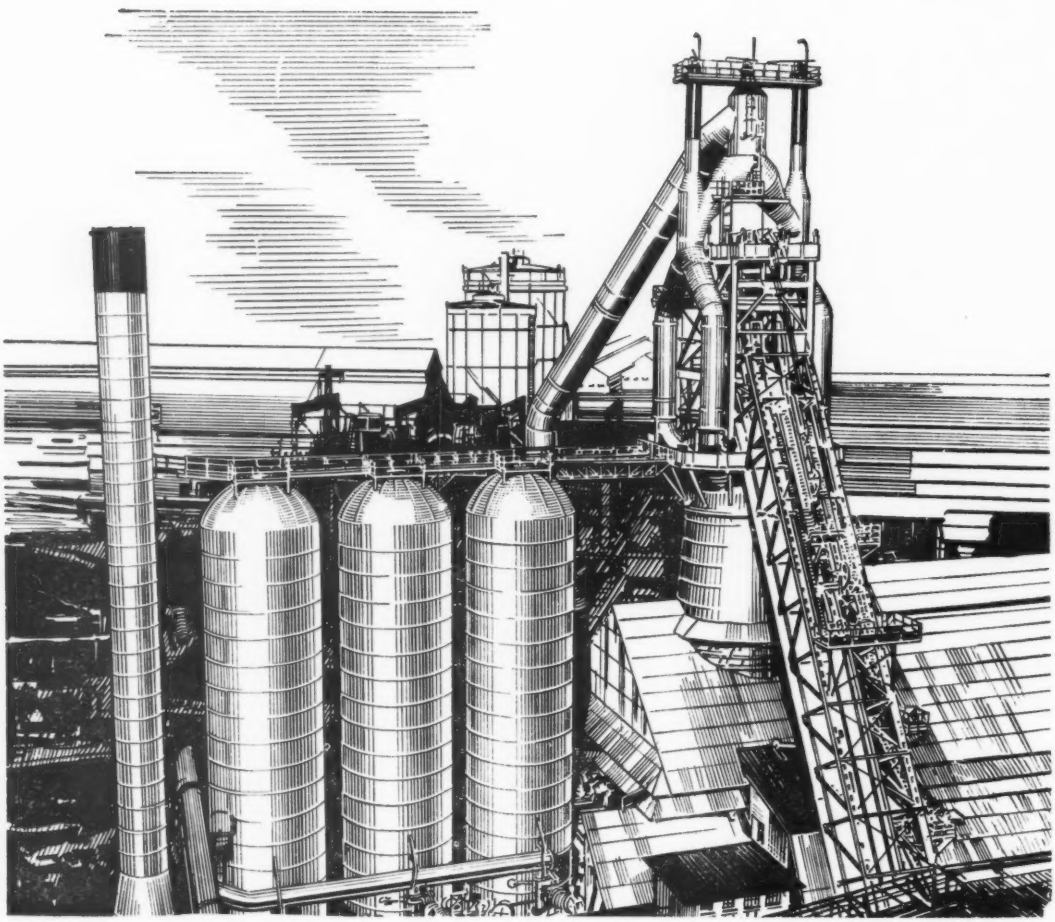

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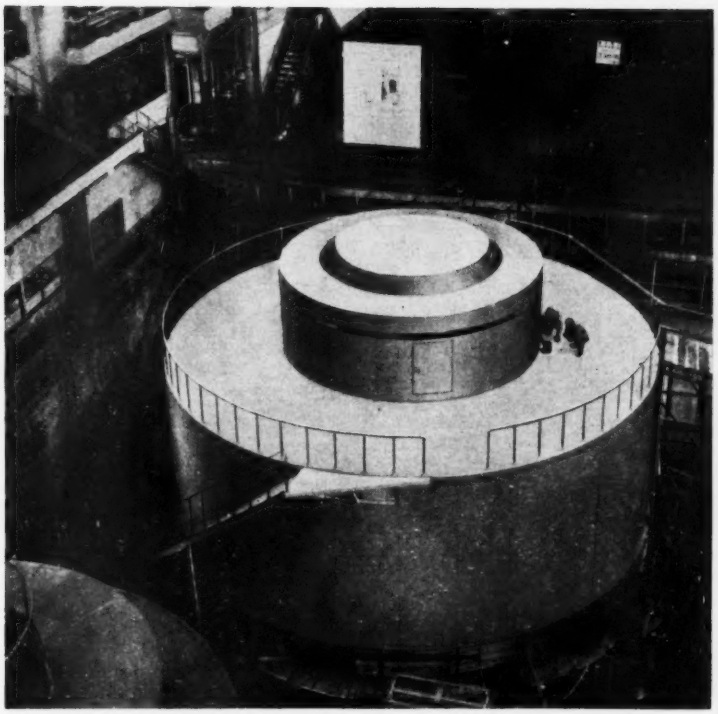

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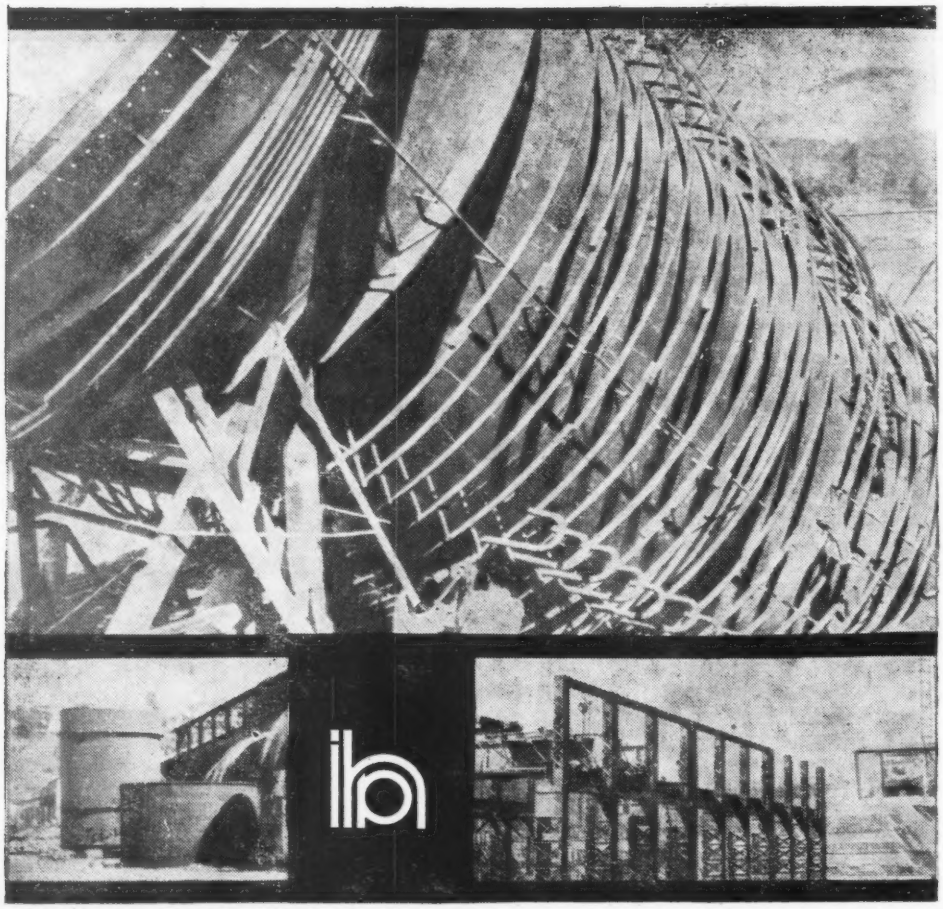

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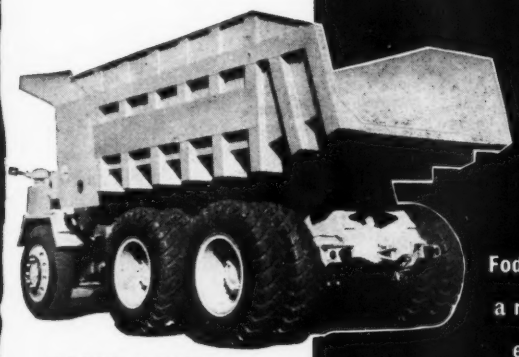

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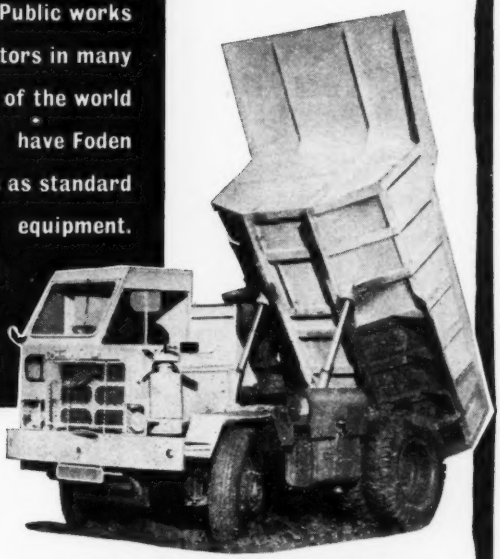

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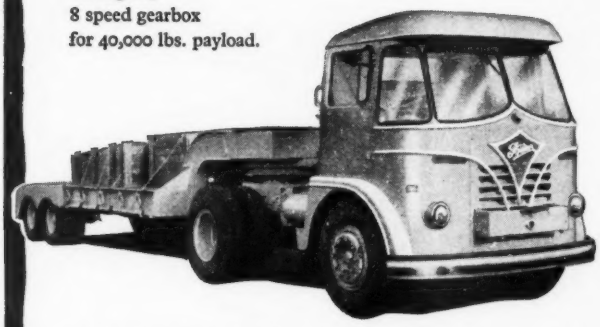

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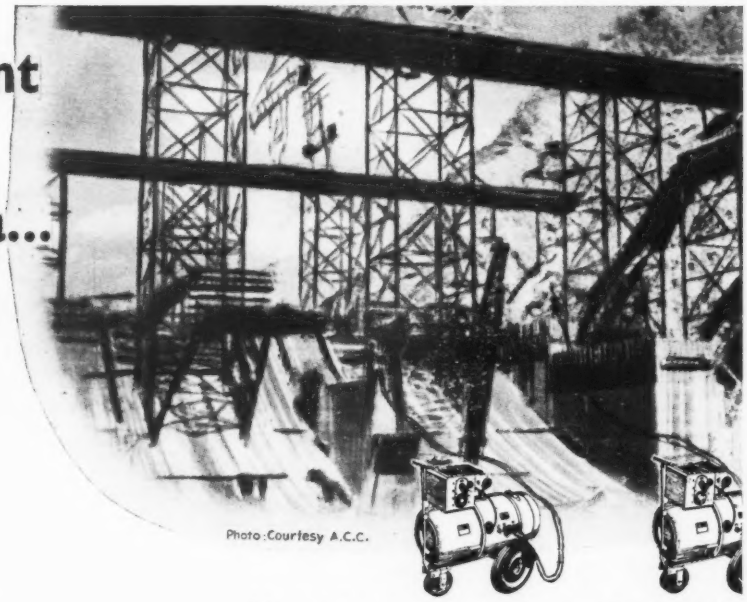

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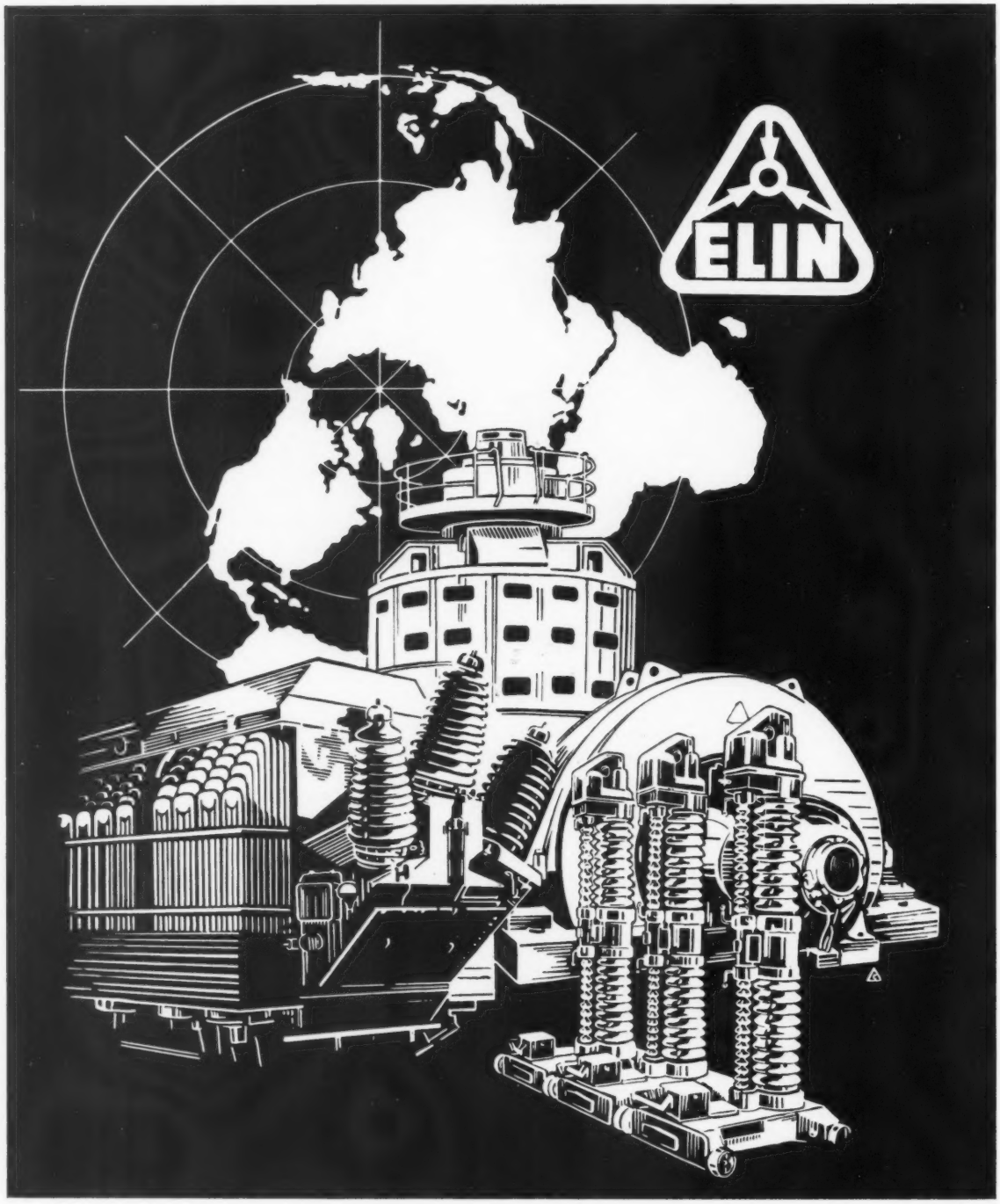

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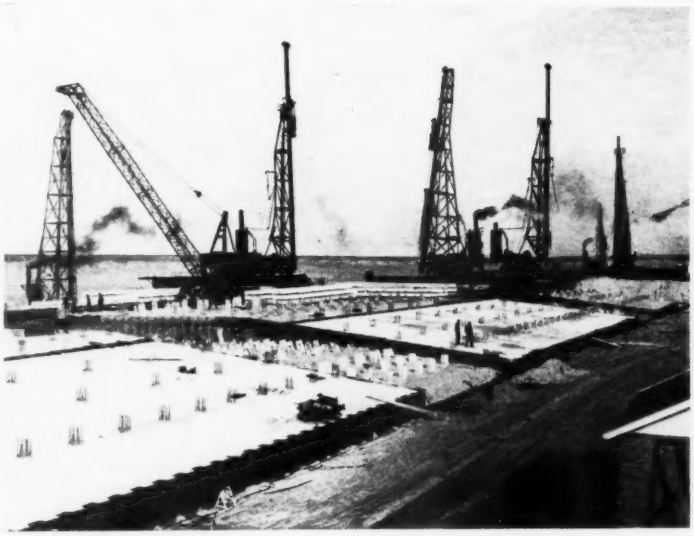

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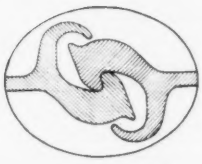

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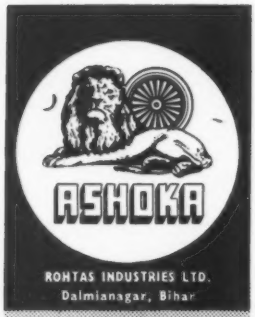

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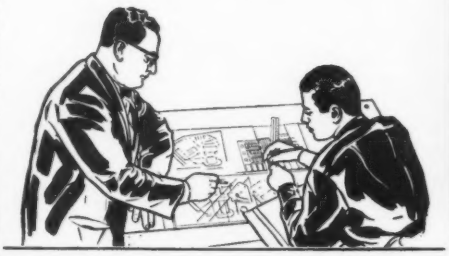

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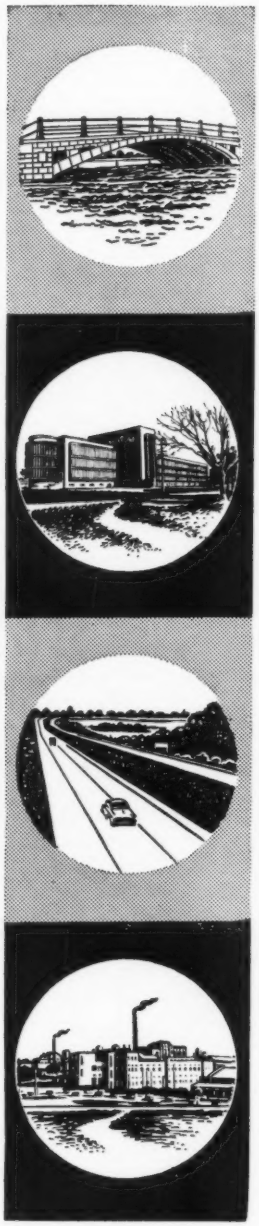

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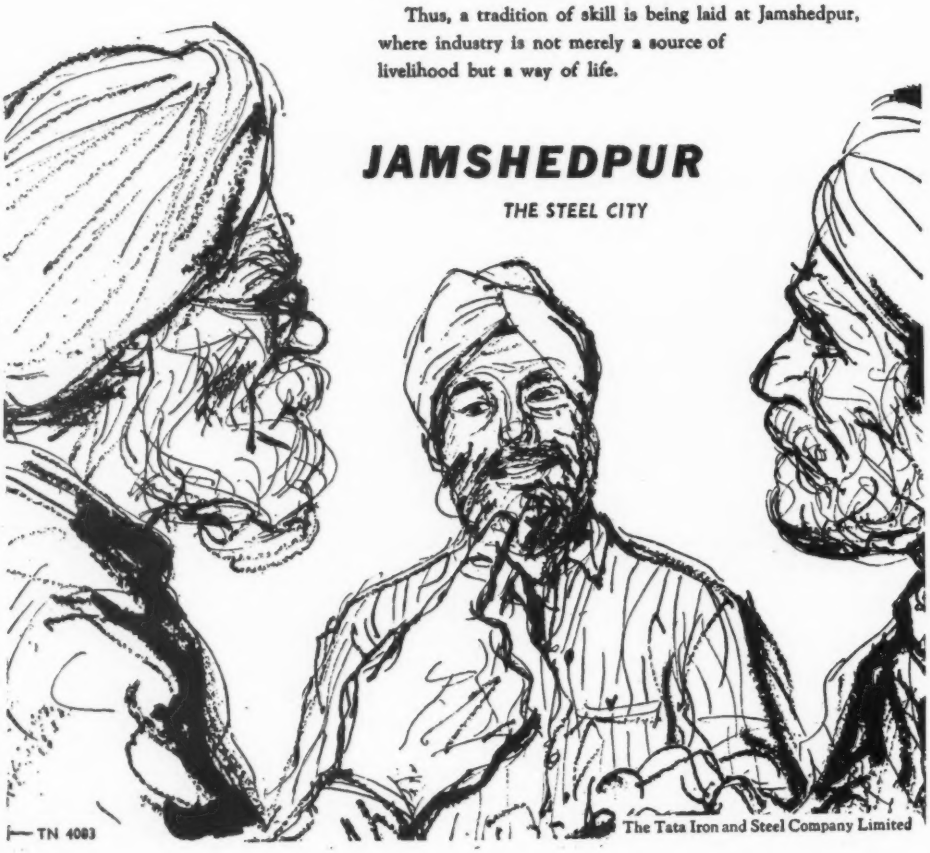

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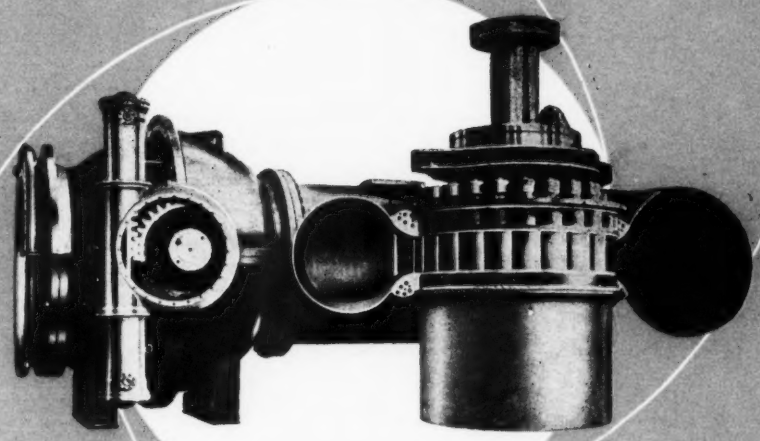

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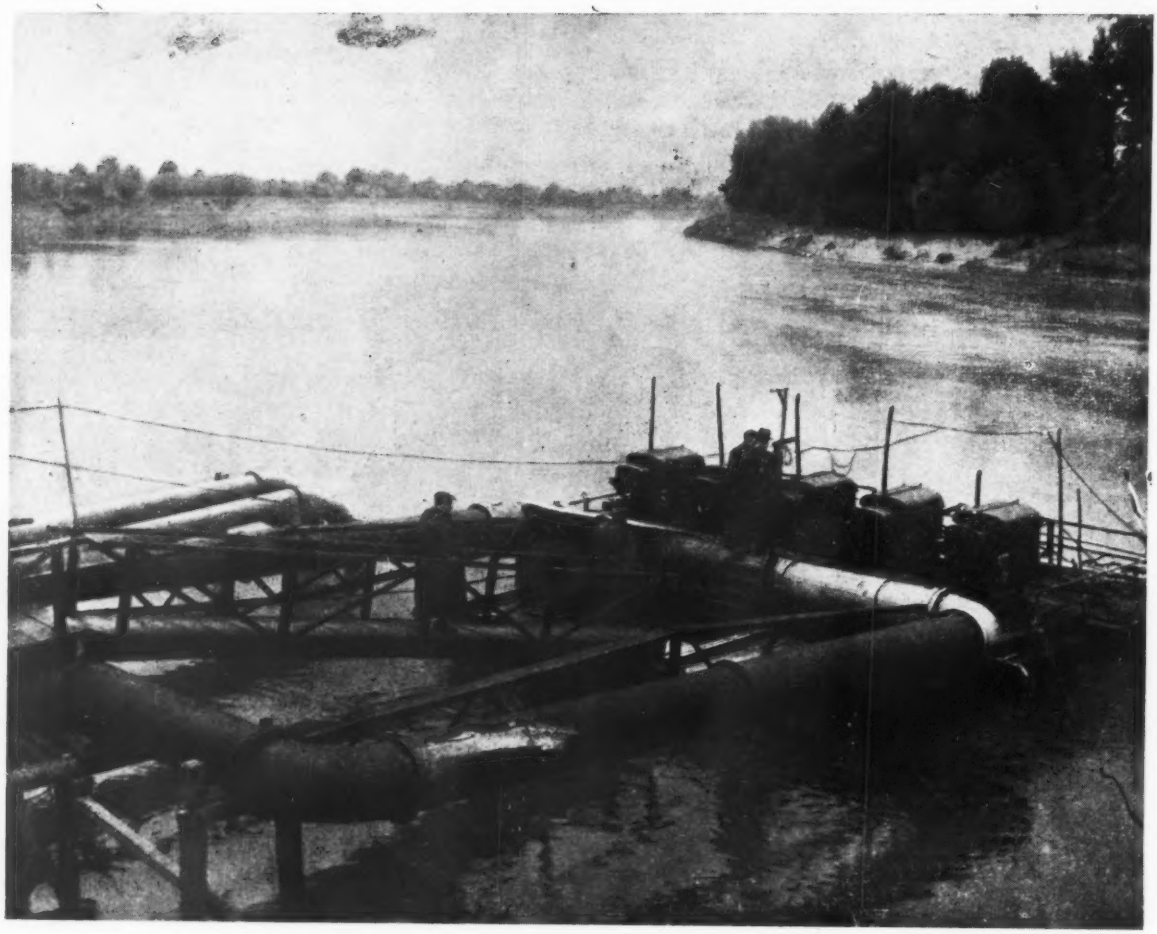

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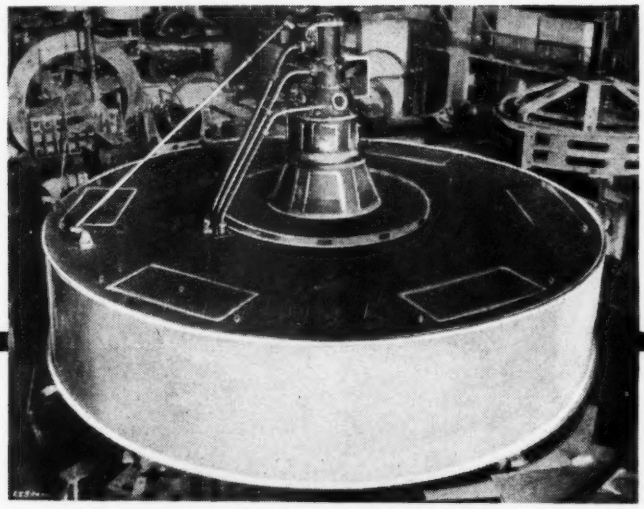

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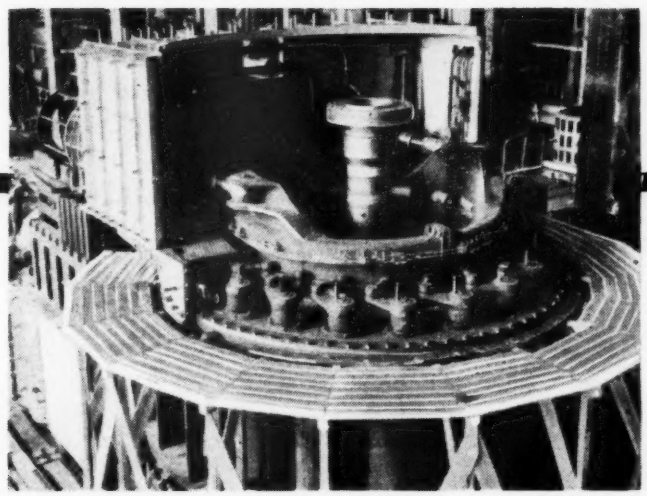

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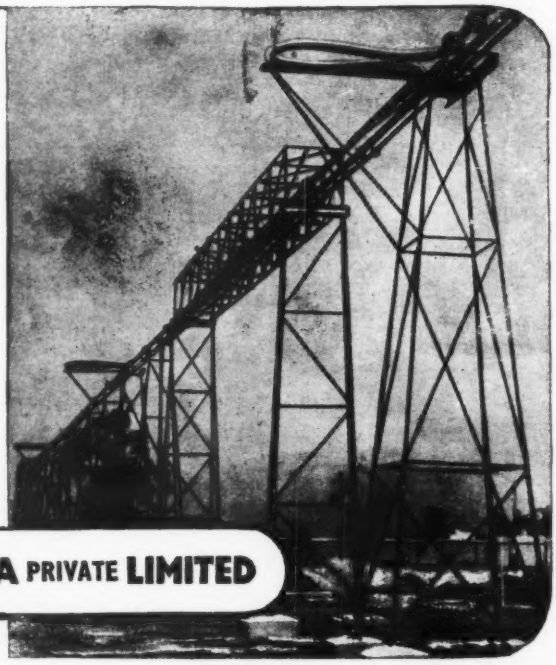

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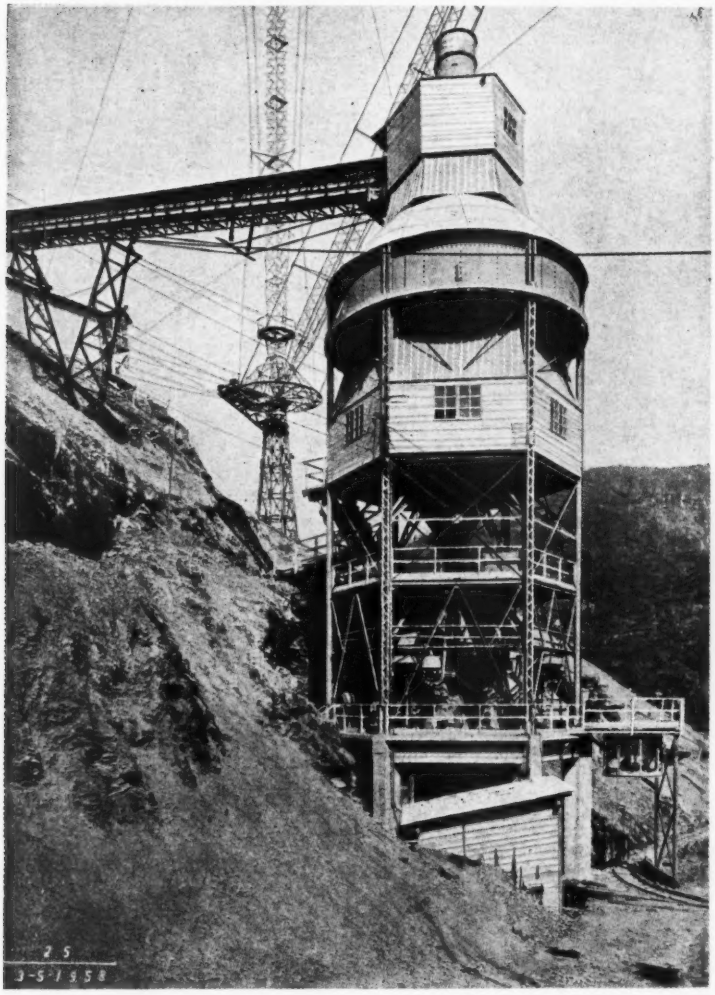

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RIHAND PROJECT NUMBER

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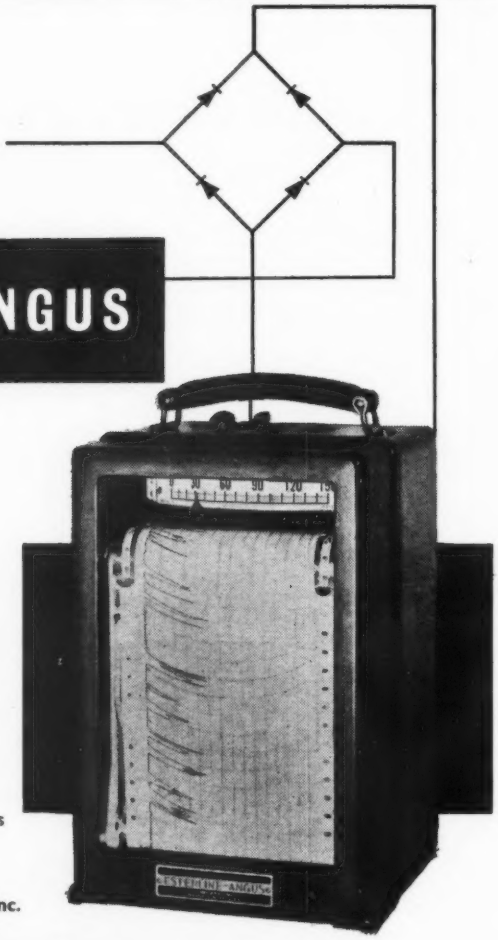

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The spark has been given to my engineers and others to go ahead with this project. May they do their work speedily and well and build up a monument for today as well as for future generations!

Jawaharlal Nehru
July 12. 1954.

Messages

The Indian Journal of Power and River Valley Development are doing a real service by bringing out a Special Number in January 1960 devoted exclusively to the Rihand Dam Project. The Project situated in proverbially the most backward and poorest tracts of our State is expected on its completion to change the very face of the south-eastern areas of the State. No country in the modern age can have any claim to advancement without adequate industrialisation and for this adequate power is the first requirement. With the generation of electricity, the entire area will by and by be streamlined with heavy, basic and small industries bringing employment to lacs and lacs of people, thus solving the problem of their bread and butter. The salvation of our country lies in the successful completion of projects like Rihand and I congratulate all those working towards this end. Posterity will always remember them with gratitude for their ennobling work.

V. V. GIRI
Governor, Uttar Pradesh

Having been closely associated with the planning and progress of the Rihand Dam Project from the very beginning, I am very happy to learn that the Indian Journal of Power and River Valley Development is bringing out a Special Number on the Rihand Dam Project in January, 1960. I congratulate the Journal on this effort of theirs. Such a venture, I am sure, would not merely be useful to engineers and administrators but also to the general public in enabling them to understand and realise the benefits which would accrue from the river valley projects such as the Rihand Dam Project.

HAFIZ MOHD. IBRAHIM
Minister for Irrigation and Power, Government of India

The Rihand Dam Project, which the Uttar Pradesh Government have undertaken, is likely, on completion, to bring about considerable prosperity in an area which formerly was among the most backward in the State. Already this lone and desolate corner of the State is humming with activity which, however, will not cease with the completion of the Dam. It is a matter of considerable satisfaction to the State Government that a large Aluminium Factory is going to be set up in that area and this is sure to give a permanent fillip to the economy not only of the Mirzapur district but also of other districts in the south-eastern portion of the State. The importance of the Rihand Project in the economy and prosperity of Uttar Pradesh cannot be over-emphasised and, for that reason, I am very glad to find that the Indian Journal of Power & River Valley Development has decided to bring out a Special Number dedicated to this Project.

The Engineers of Uttar Pradesh have had to face a number of intricate technical problems in the execution of this Project and it was a challenge to their ingenuity and skill to find solutions for difficulties the kind of which they had never encountered before. I welcome this Special Number of the Indian Journal of Power & River Valley Development also as a tribute to the very fine work which our Engineers are performing in a remote and out of the way corner of the State under conditions of work and living which are by no means easy.

(DR.) SAMPURNANAND
Chief Minister, Uttar Pradesh

I am glad to note that the Indian Journal of Power and River Valley Development, Calcutta propose to bring out another Special Number, this time on the Rihand Dam Project. The Project would go a long way in relieving the acute shortage of power in that State.

My best wishes to the Journal for publishing a Special Number devoted to this Project.

J. L. HATHI
Dy. Minister, Irrigation and Power, Government of India

Messages

I am glad to note that the Indian Journal of Power & River Valley Development is bringing out a Special Number, entirely devoted to Rihand Dam Project, a premier hydro-electric project in Uttar Pradesh. The Journal has been rendering yeoman service in disseminating very useful information about the development of power and river valley projects and is being eagerly received by both engineers and the public. I am sure this Special Number is also going to be of considerable use to all concerned. I wish the Journal a grand success.

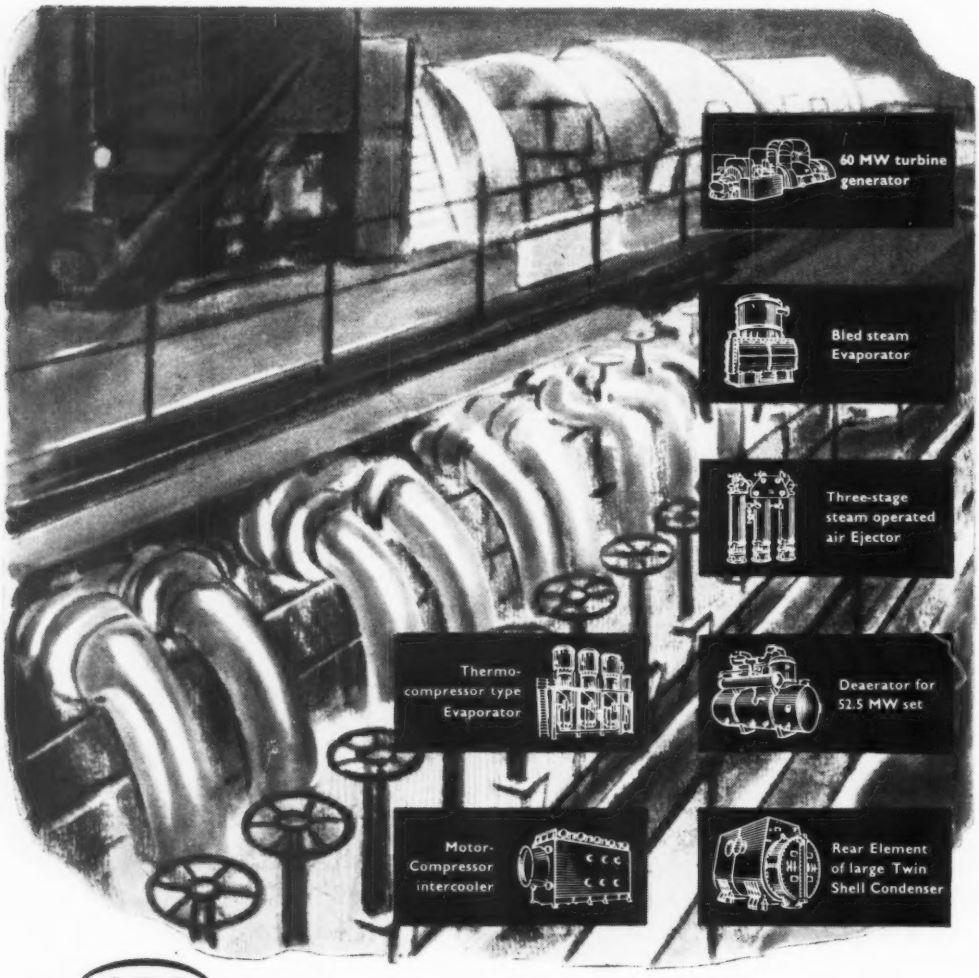
M. HAYATH
Chairman,
Central Water and Power Commission

INTRODUCTION

The special issue is devoted to yet another major valley project in the country. The Rihand Hydro-electric Project is situated in a region which has remained undeveloped for quite a long time, although adjacent areas have been making progress at a comparatively faster pace. Yet, with possibilities of industries like chemicals, metallurgy, aluminium, fertilisers, porcelain, paper etc, some of which are already under way, this region has a richer potentiality. Now that hydro-power from Rihand will be available in substantial quantities soon, it will not take much time to cover the leeway in progress.

But hydro-power will not be enough, as it will need to be firmed up. That is why, even while the dam rises, there are big plans for a major thermal station at Singrauli, with a capacity of 450,000 kW. The two together, and in association with the DVC and Madhya Pradesh Power Systems, and other thermal power stations such as at Kanpur and Lucknow, and the Ganga and the Sarda grids, the Rihand grid will become the life-line of feverish industrial activity in the entire region. In the words of the Prime Minister, who has correctly gauged the mood of the people, a dream will have come true.

But a modern dream to be realised in actual life and experience of the people needs intensive and accurate planning, and complex but competent execution. In this respect the engineers-in-charge deserve our fullest encouragement. It is also gratifying to note that the chief contractors have given adequate proof of their technical capacity against several handicaps. The following contributions give not only the technical details of the many facets of the project under construction, but also of the features of its construction. We are sure this will provide an adequate and authoritative reference material for the project as a whole.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Pioneering of Rihand Dam Project

By A. C. MITRA

AFTER the first World War, a reconnaissance of the hydroelectric power potential in various river valleys of India was carried out between the years 1919 and 1923 and it was then considered feasible to conserve the energies of Sone river and its tributaries for power generation. No sites for the construction of dams and power houses were however located nor was any project prepared.

About the same period, the development of power potential of the falls on the Upper Ganga Canal was commenced and a series of low head hydro power stations were constructed. By the year 1935, the Ganga hydroelectric grid was established to serve the western districts of Uttar Pradesh. The benefits, which accrued to the people of these western parts of the pradesh, naturally led to the dam and for establishment of a similar eastern hydroelectric grid for the development of the backward eastern districts of Uttar Pradesh. Shri A. P. Watal I.S.E., then Executive Engineer was placed in charge of the surveys and investigations for the purpose.

The sites for hydro power generation are usually located in remote places in hilly terrain full of jungle and having little or no means of communication. The valleys of river Sone and its tributaries were all the more difficult as this part of Uttar Pradesh and neighbouring States was, till then, very backward and almost trackless and sparsely populated. In the year 1936-37 Shri A. P. Watal, however, ventured to trek down all the distance from Robertsganj across the unbridged river Sone and then up the Rihand river Valley, for several days, carrying whatever he could for the daily necessities of life and drinking water from the road-side ponds. He heaved a well-earned sigh of relief and joy when he happened to locate this narrow gorge through which the Rihand river passes about twenty miles upstream of its confluence with river Sone. The site is ideal for a high dam and looks as if it was made to order for the construction of a dam. It has exposed sound granite rock all along the bed and has a very wide valley on the upstream affording excellent storage capacity for a comparatively low height of dam. The detailed surveys later revealed that the storage for a 300 ft. high dam would be 8.6 million acre ft.

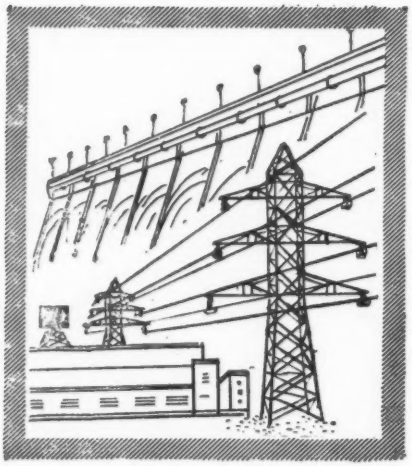
Shri A. P. WATAL

However, this very promising project was not to go through as yet, as the Second World War commenced soon after the discovery of the site.

By the time the war was over Shri A. P. Watal rose to the post of Superintending Engineer, Development and, in the year 1943, he undertook the survey of the site and the Rihand Valley in detail and a preliminary project was prepared by him envisaging the construction of a 300 ft. high concrete dam, which was considered a bold step at the time as dam of this height were then rare in India. The project however, was not yet to be translated from the blueprints on to the wonderful gorge, in concrete and steel and was shelved once again, as greater priority had to be alloted to short term grow-more-food schemes in an attempt to mitigate the serious food shortage in this country.

The first Five Year Plan, brought the project into lime-light again and sub-surface explorations in greater detail were commenced in the year 1952 with a view to commence construction of the Project but the progress continued to be very slow until the year 1954 when the Technical Cooperation Mission of U.S.A. came forward to help the Project and offered to make available all the foreign exchange

(Continued on page 8)



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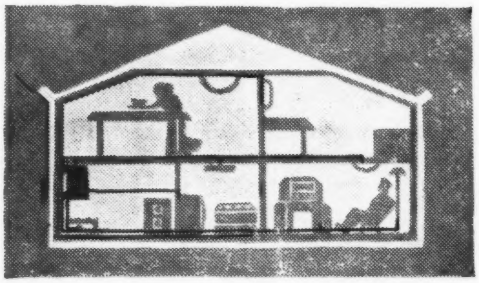

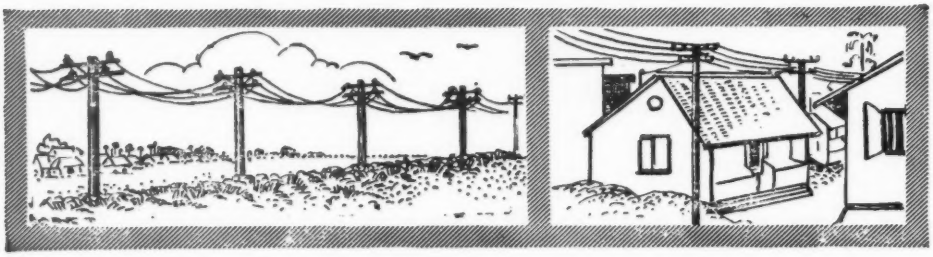

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Rihand Hydro-Electric Project and Power Development in Uttar Pradesh

By K. C. GUPTA

UTTAR Pradesh, the land of sacred rivers Ganga and Yamuna, from whose waters millions derive spiritual solace, is also blessed with resources which can be developed to generate electricity. One such development began about 35 years ago when the water of the Ganga canal, which till then irrigated farms and fields, began to be harnessed for the production of power also. Thus a series of hydroelectric generating stations was established on the Ganga canal, which, between them, have a total generating capacity of 45,200 kW. This power is distributed in the western Uttar Pradesh and it furnishes light in houses, turns the wheel of industry and drives tube-well pumps so that fields beyond the reach of the canals may also share in the prosperity which irrigation brings. Thus the foundation has been laid of a power system which has the unique feature of generating hydro power from canal water which, in the past, was utilised only for irrigation purposes.

Nearly 15 years ago decision was taken to harness the water of the Sarda canal and a 41,400 kW hydro-electric generating station was set up over this canal at Khatima. This power house serves the northern part of central Uttar Pradesh and supplements the power system at Lucknow. To meet the needs of the present day and of the immediate future—needs which, in a developing economy, are appropriately, on a very much vaster scale than was thought of in the past—a 250,000kW hydroelectric scheme, the Rihand Project, is taking shape in the south-eastern part of the State. The power house will be located at Pipri on the river Rihand, a tributary of the Sone. Unlike the power projects on the Ganga or the Sarda canals, this project will require construction of a 271 ft. high, concrete gravity dam, 3,254 ft. long,—one of the biggest in India. This dam will be located in the area so closely associated with the God of Engineering, Vishwakarma, of Pauranik age; and this seems a good augury for the rapid development of industries which is expected in this part of the State.

The water to be impounded behind the dam, which is rapidly nearing completion, will create a lake of no less than 180 sq. miles in area having a storage capacity of 8,600,000 acre ft. This lake will be filled up during the monsoons and water therefrom will be available throughout the year for the production of power and, what will be no less important, for irrigation purposes in the areas commanded by the lower reaches of the river Sone. It is an interesting fact that whenever a river runs naturally and unrestrained, its water during the monsoons is of little value for irrigation, and when in heavy flood, it can spread loss and suffering, and during the dry seasons the water is insufficient to meet the needs of irrigation. On the other hand, when the river is restrained by the construction of a dam, its flow is regulated and the disadvantages of unrestrained flow disappear. The water stored behind the dam is used for production of power and by reason of its flow being controlled, it can be utilised for irrigation purposes and the waste due to unrestrained flow of flood waters to the sea is eliminated.

That, in brief, is the objective for the construction of the Rihand hydro-electric project. Power from this project will be used for electric traction for the State-owned cement factory at Churk in District Mirzapur, for industrial purposes, such as, the factory for the production of aluminium at Pipri which, incidentally, will be the largest in India, for lighting houses and for miscellaneous purposes.

The production of aluminium at Pipri will be a great asset. Not only will it save foreign exchange to the extent of about Rs. 5 crores or so spent annually on the importation of this valuable metal, but it will also aid in the setting up of subsidiary industries producing articles made of aluminium, thus providing skilled and unskilled employment to thousands of work people. As a measure of demand for aluminium, it may be mentioned that even while the original project is in its preliminary stage, steps are being taken to increase its production capacity to two and a half times the level originally proposed. The existing demand for power in the State is so great that the entire production from the 250,000 kW project has already been fully booked by prospective consumers and more power is urgently required. Fortunately, adequate deposits of coal have been discovered in the neighbourhood of Pipri of quality suitable for generating electricity. It has, therefore, been decided to establish a thermal power house of

450,000 kW capacity near the coal fields south-west of Pipri. The Rihand area is rich in mineral wealth also. Marble, granite, lime stone, mica, bauxite and corundum are amongst the minerals which occur in quantities sufficient for economical development.

Thus the natural resources of the Rihand area will be developed for production of power where it can be produced most economically. This power will be transmitted by means of a 220,000 volt super-grid to the central, western and north-western Uttar Pradesh and along its route it will link up with the major power houses at Kanpur, Harduaganj and eventually with the hydroelectric power houses to be constructed for the utilisation of the hydroelectric potential of the Yamuna and Ramganga rivers in the north-west of Uttar Pradesh. The Rihand project in the south-east of Uttad Pradesh thus becomes the starting point of a major power system which will spread itself right across the State reaching up to its north-west areas, serving people all along its route and across by means of subsidiary lines. The years immediately ahead, thus, bear a rich promise of harnessing the bounties nature has provided in the shape of hydroelectric resources and mineral wealth for the economic betterment, prosperity and happiness of the people.

(Continued from page 6a)

PIONEERING OF RIHAND DAM PROJECT

required for the construction plant for Rihand Dam, and also any technical help required. Thereafter, global tenders were invited for the construction of the Dam and appurtenant works and the contract was awarded to Messrs. Hindustan Construction Co. Ltd., in April 1955.

The construction plant was set up during the course of the next two years and the first bucket of concrete was laid in April 1957. Since then, the work is progressing round the clock and according to schedule—in fact a little in advance of schedule. Power is expected to be available from the project by the end of the year 1961.

The availability of this large block of power (250,000 kilowatt installed capacity) at economical rates is expected to result in rapid industrialisation of the State and help in supplying electric power to the economically backward eastern and south-eastern parts of the State. In fact all the Rihand Project water has already been earmarked and investigations have already commenced to establish more hydro and thermal power stations in this and other areas of this State.

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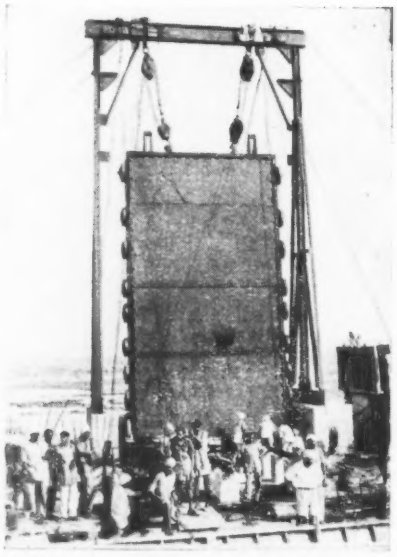

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Geology of the Rihand Dam Project

By G. S. M. RAO

RIGHT since the inception of the Rihand Dam Project in Uttar Pradesh in 1943, the geologists of the Geological Survey of India have been in-charge of the geological investigations and have been apprising the engineers of the various geological factors influencing the design and construction of this major hydro-electric structure in U.P. which would generate 2,40,000 kW.

In this article it is proposed to review the geological features influencing the sequential development of the project as from planning, site selection, preliminary and detailed design stages to the stage of construction now under way.

Shri D. R. S. Mehta, Geological Survey of India, after a preliminary survey during 1943, suggested the shifting of the alignment of the dam from the zone of Bijawar phyllites, to the zone of gneissose granite situated about a mile further downstream of Pipri village. Later, between 1946 and 1952, Shri S. P. Nautiyal formulated the exploratory programme for the final selection of a suitable alignment for the dam within the 600 ft. wide band of the gneissose granite, and for the assessment of construction materials. He was assisted in this work by Sarvashri G. Kohli, S. C. Awasthi, M. S. Jain, B. P. Tewari and S. S. Misra of the Geological Survey of India, and Shri B. R. N. Murthi, a geologist of the Rihand Dam Project appointed by U.P. Government. Since 1954, the writer has been associated as the Resident Geologist-in-charge with the investigations relating to the design and construction stages. He was assisted by Shri R. S. Verma between 1954 and 1956 and by Shri B. L. Gupta of the Rihand Dam Project since 1957.

The geological investigations have been carried out under the guidance of Dr. J. B. Auden between 1944 and 1952, and Dr. P. K. Ghosh between 1953 and 1955 and later under the supervision of Shri P. C. Hazra.

LOCATION

The 250-foot high Rihand Dam under construction, across Rihand river, a tributary of the Sone, is located near Pipri village (Lat. 24°12′30″: Long. 83°0′30″) in the Duddhi *Tahsil*, Mirzapur District. It is about 100 miles south of Mirzapur and is accessible by metalled road.

Recently Churk, which is about 48 miles north of the dam-site, where the U.P. Government Cement Factory is located, has been connected by a broad-gauge-railway with Chunal station on the Calcutta-Delhi chord line. The proposal for extending this railway up to Garhwa Road in Palamau District. Bihar, passing through Pipri has been recently sanctioned.

PROJECT FEATURES

1. Type of dam	...	Straight gravity concrete structure
2. Height of the dam	...	250 feet above the river bed level
3. Length of the dam	...	3064 feet
4. Base line of dam	...	2009 feet straight
456′—0″ — Right abutment curve: Radius 1800′; Angle at centre 20°5′51″		
600′—0″ — Left abutment curve: Radius 750′; Angle at centre 45°50′12″.		
5. Length of the spillway	...	624 feet
6. Length of the East Non-overflow section	...	1350 feet
7. Length of the West Non-overflow section	...	690 feet
8. Intake	...	400 feet
9. Bed level	...	R.L. 630
10. Top of dam	...	R.L. 880
11. Crest elevation of the spillway	...	R.L. 852
12. Gross storage at Elev. 880	...	8,600,000 acre ft.
13. Surface area of the reservoir	...	180 sq. miles
14. Catchment area of the Rihand above the dam-site	...	5,000 sq. miles
15. Annual run-off average yield	...	6,060,000 Acre ft.
16. Average annual rainfall	...	55 inches.
17. Maximum discharge considered for spillway design	...	5.7 lacs. cusecs.

Starting from the right abutment, the dam comprises of 61 independent blocks of variable widths located side by side with ungrouted joints.

GEOLOGY OF THE DAM-SITE AND THE RESERVOIR

The country rocks belong to the Bijawars (Lower-Cuddapahs) and the transition series. The southern half of the reservoir is mostly composed of the gneisses of the transition stage. A few outcrops of the Talchir Boulder-bed and shales of the Lower Gondwanas are also found on either bank of the Rihand river, about 2 miles upstream of the dam-site, which gradually broadens out further West. A fringe of

Barakar formations, comprising mainly sandstones and shales, extends 2 to 4 miles into the reservoir towards its north-western boundary (Plate I).

The following sequence of rock formations are met with at the dam-site and its vicinity, arranged in order of superposition:—

5. Quartzites (mostly of the nature of quartz-breccia)
4. Amphibolites
3. Phyllites and schists
2. Injection gneisses
1. Granites

For limiting the size of this article it is felt that only a brief review of the characteristics of the formations, which form the foundation of the dam, would be adequate for understanding their implications in the design and construction of the dam.

Granites

These include massive gneissose granites, aplites and quartz-veins together with thinly foliated gneissose granites.

Massive gneissose granites constitute the major portion of the foundation of the river-bed and abutments of the proposed dam. These being quite compact and hard, are resistant to weathering, except where the gneissose texture is well developed.

Thinly foliated gneissose granites: They are almost similar to the massive gneissose granites except for their foliated nature and variation in the crystallinity resulting from intense shearing. Such types are found in the right abutment between R.L. 660 and 720 and above R.L. 800.

Alternating bands of massive and thinly foliated gneissose granites are found on the upstream side of the dam with their strike varying from E.-W. to E.S.E.-W.N.W. with dips at 50°-80° towards south.

The gneissose granite is greyish in colour and coarse to medium grained in texture. It generally trends 110°-290° and has developed pseudo-bedding planes with dips varying from 70°-80° towards south. The dynamic metamorphism has resulted in the development of numerous joints of which those with N.W.-S.E., N.N.E.-S.S.W., and E.S.E.-W.N.W. trends are most pronounced. Prominent sheeting and shearing are seen along some of the joints of which those trending N.E.-S.W. are most conspicuous. Shearing along the joints, which is responsible for deep weathering, is seen in the abutments. Slick fractures, seen along some of the joints, are indicative of movements and minor slips of the rock mass.

Injection Gneisses

These represent the granitised slate and phyllites of the area and exhibit the characteristic banding resulting from lit-par-lit injection. These have an E.-W. trend with dips of 65°-75° towards south. Local foldings and flextures are apparent at places.

The contact of the injection-gneiss with the gneissose granite is clearly seen towards the downstream toe of the dam. These formations strike 105°-285° and dip at 60° towards upstream at the contact. Some shearing along the contact is seen at the power-house site.

In addition to the above two major formations, minor bands of phyllites, quartz-chists, mica-schists, amphibolites, aplites and quartz-veins are also encountered in the foundation area of the dam.

PLANNING STAGE INVESTIGATIONS

Investigation of the Dam-site and Reservoir area: The reservoir, spreading over an area of 180 sq. miles, has been geologically mapped on one inch to one mile scale. The surveys did not reveal occurrence of any major mineral deposits of economic value within the reservoir. Possibility of any leakage from the reservoir has also been ruled out. However, the scope for any enlargement of the reservoir by the construction of a higher dam is restricted by the occurrence of the Kota coal-fields at the reservoir rim, which, if submerged, would deprive the country of a valuable source of medium grade coal, and the corrundum deposits at Pipra in Madhya Pradesh.

SITE-SELECTION STAGE INVESTIGATIONS

During 1944, after a geological reconnaissance, the shifting of the proposed dam alignment from the zone of Bijawar phyllites, further downstream, to the 600-foot wide gneissose granite band within the gorge was suggested. The gneissose granite band, striking E.-W. across the river, was found to be closely jointed and foliated at places due to the intense regional stress which resulted in the development of pseudo-bedding planes dipping at 70°-80° towards the uptream (south and south-west). The close jointing combined with the foliation and shearing of certain zones accentuated deep weathering at places in an otherwise fresh and sound granitic terrain. Hence, to locate the most economical alignment, based on the depth to sound rock, existence of weak zones, contact of the different rock types and the foundation treatment necessary, no less than seven different alignments between Kukri Howa hillock on the left bank of Rihand and the hillock on the opposite bank were explored by drill holes, trenches and test pits.

On the basis of relative economics, the alignment suggested by Dr. J. L. Savage, Consulting Engineer from the U.S.A., was accepted in 1948. This alignment has a N89° 10′W trend in the river bed portion which curves downstreamwards above 700 ft. contour at the left abutment, and above 780 ft. contour at the right abutment. These curves, being transverse to the topographical contours, would reduce the length of the dam. This axis was explored in detail by 13 drill holes located at intervals of about 200 ft.

In the gneissose granite of the right abutment, drill hole No. 32, located at R. L. 835, indicated the depth to sound rock to be about 84 ft. and along drill hole No. 34, located at R.L. 914, sound rock was encountered at a depth of 57 ft. Hence, a composite dam with an earthen section in the highly weathered zone of the right abutment was proposed.

Owing to the paucity of suitable earth materials in the vicinity of the dam-site and also the advantage of having a strong uniform structure which would avoid the complexities involved in the construction of a composite dam, a thorough exploration of the right abutment was undertaken during 1954. This enabled the final selection of the present alignment for the construction of an all concrete dam (Plate II).

The layout of the final alignment is straight in the central portion for a length of 2009 ft. At the right abutment, the alignment, comprising a length of 456 ft., is curved with a radius equal to 1800 ft. and subtended angle equal to 20°5′51″. At the left abutment section, comprising a length of 600 ft., the alignment is curved with a radius equal to 750 ft. having a subtended angle equal to 45°50′12″.

Exploration of the dam-site for the selection of a suitable alignment involved drilling of about 110 diamond drill holes aggregating to about 7000 running feet; geological interpretation and correlation of which enabled the final selection of a suitable alignment. The detailed geological studies enabled the geologists of the G.S.I. to suggest the possibility of constructing a concrete dam after the treatment of weak zones and with a slightly modified alignment in preference to the originally proposed composite dam with an earthen section in the right abutment because of the excessive depths of weathering (about 80 ft.) of the foundation rocks.

DETAILED DESIGN STAGE INVESTIGATIONS

In order to get a correct picture of the foundation levels to be attained to reach the final foundation grade, the sub-surface geological data, obtained during the exploratory stage, were further augmented by drilling additional 70 holes aggregating to about 2400 running ft. The extent of water tightness of the foundation rocks was determined by percolation tests through about 200 drill holes totalling a footage of over 10,000 ft.

In addition to the weathering of granite, which constitutes a major defect in the foundation, the presence of open joints, shear zones, and the contact of the granite with the injection gneiss at the downstream toe of the dam, were studied in detail during foundation stripping involving removal of nearly 24 million cubic ft. of rock. The behaviour of these adverse features at depths was further studied with the help of drill cores and percolation tests which indicated that the joints and contact planes became tight at depths. Grouting along critical zones indicated that while the joints are tight at depth and consumed very little grout, the shear zones required appreciable grouting indicating their openness. The geological features of the different sections, which influenced the design of the dam, are briefly summarised below.

Eastern Non-Overflow Section

This section starts from the right abutment and comprises blocks 1 to 27 covering a length of 1350 ft. of the dam. In this region, altogether 54 holes have been drilled from which the probable sound rock contours have been interpolated. The area is mostly composed of gneissose granites and injection gneisses with minor pockets of phyllites, mica-and chlorite-schists found as inclusions within the granites. Two sets of joints, one dipping at 25°-30° upstream and the other at about 20° downstream, are prominent. In addition to these, there are vertical joints with sheared and crushed zones varying in width from 2 to 8 ft. These are very prominent, above R.L. 800, on account of which the weathering pattern has been very irregular. Often shearing along the joints has resulted in the formation of seams of weathered rock of varying thickness below fresh rock. Such seams are exposed in the excavated foundation of blocks 19 to 26.

In view of the above nature and disposition of the weathered zones, it was suggested that a specification governing foundation stripping may be laid down indicating the special circumstances in which some of the weathered rock could remain in the foundation of the dam.

At the right abutment, stripping in certain zones upto the sound rock was found to be uneconomical because of the deep weathering of granite. Moreover, in view of the very low height of the dam in this section, it was felt by the project engineers that very deep excavation for the ideal foundation grade was not necessary. Moreover, the weathered granite had indicated a crushing strength of about 6000 lbs. per sq. inch after 72 hours of soaking and the maximum stress in this section of the dam is not likely to be more than about 70 lbs. per sq. inch. Therefore, it was decided by the engineers that excavation upto the fresh rock was unnecessary.

Along zones, where weathered rock underlies the fresh rock, it was decided that the weathered rock should be excavated down to twice its thickness and then back-filled with chilled concrete. Where necessary, further consolidation was proposed by grouting through closely spaced drill holes after thorough washing.

In blocks nos. 18 and 26, the injection-gneiss is found to extend about 8 to 40 ft. south of the downstream toe of the dam. It has numerous prominent pockets of biotite-chlorite-schist and amphibolite. Between blocks 23 and 26, the amphibolite appears to have been much sheared and is almost chloritic.

Numerous stringers of quartz-veins are found here with disseminations of chalcopyrite and pyrite.

Most of the vein-quartz, encountered in the area, have a shattered appearance. This feature together with the occurrence of the slick fractures along some of the joint planes, suggest the possibility of there being relative movements and minor slips of the rock formations, accompanying a fault whose existence in the vicinity of the dam-site is suspected by the existence of shear zones, faulted quartz-veins and slicken sides.

Final foundation levels of the Eastern non-overflow section would vary between R.L. 830 and R.L. 610.

The power house foundation area comprises a length of 400 ft. covering blocks nos. 28 to 33. Here too the gneissose granites and injection-gneisses with minor bands of amphibolites and mica-schists form the rock types. Here, portions of the amphibolites and mica-schists have been rendered very soft and have an almost clayey appearance due to intense shearing and weathering. This zone appears to taper off towards the downstream western edge of block 30. In this area, 13 holes were drilled which have indicated that the sound rock levels in this zone vary between R.L. 633 and 598.

The foundation of the spillway section of the dam comprises a length of 624 ft. extending from block no. 34 to 46. Gneissose granite is the main rock type. The injection-gneiss is found to extend at the downstream end of blocks 34 to 38. Between blocks 34 and 24, several deeply weathered zones are found, which may be due to fluctuations in the river flow. Pockets of biotite-and chlorite-schists and phyllites are also found within the gneissose granites.

The outcrops of gneissose granites in the perennial channel are spheroidal and rugged because of numerous large-sized pot holes. In the foundation of the spillway 43 holes have been drilled which indicated that the final foundation levels vary between R.L. 641 and 600.

Along block 34, which was originally the perennial channel until the river was diverted to flow through block 25 in March, 1956, large outcrops of fresh massive granite were seen. From their surface examination, it had earlier been thought that no excavation would be necessary in the river bed portion except for purposes of anchorage. These exposures of granites have been affected by horizontal joints which have promoted weathering of the underlying rock. Three drill holes located along the block indicated the presence of sound rock at depth varying from 8 to 20 ft. thereby necessitating excavation in the river bed portion also.

Of the 43 holes that were drilled in the spillway portion of the dam, 12 revealed high leakage during percolation tests, indicating that there are potential zones of seepage which should be successfully sealed off by grouting.

Western Non-Overflow Section

This comprises a length of 690 feet, extending from block no. 47 to 60 and terminating at R.L. 880 on the left abutment. Gneissose granite comprises the major portion of the foundation. Inclusions of squartz-schist are found within the gneissose granite. These are thinly foliated, highly disturbed and are in places partially granitised.

Closely spaced vertical joints are prominent above R.L. 700. Weathered clayey seams of thickness varying from 2″ to 10″ are found along joints and fractures intercepting the vertical joints and dipping at angles varying from 25° to 40° towards downstream.

The dam would be founded on granite while the power house would be founded on the injection-gneiss. . The junction between the granite and the injection-gneiss is discontinuously sheared. Exploratory drilling revealed that the shearing dies out at R.L. 590, *i.e.*, about 40 ft. below the original river bed level. In addition to the shear zone at the contact of the granite with the injection gneiss, there are 7 prominent cross shear zones extending from the axis to the toe of the dam. These dip at an angle of 55° to the south and southwest. These shear zones have influenced the final foundation levels and have received special attention in foundation treatment to guard against the possibility of their acting as conduits for the seepage of the reservoir water below the dam. The Rihand dam-site lies within the isoseismals VII and VIII of the Bihar-Nepal Earthquake of 1934. Hence, a seismic factor of .1 g was recommended to be adopted in the designs.

CONSTRUCTION STAGE INVESTIGATIONS

Due to favourable geological conditions, general consolidation grouting was found to be unnecessary and only the discontinuous thin shear zones, observed at the axis of the dam and at the toe dipping at low

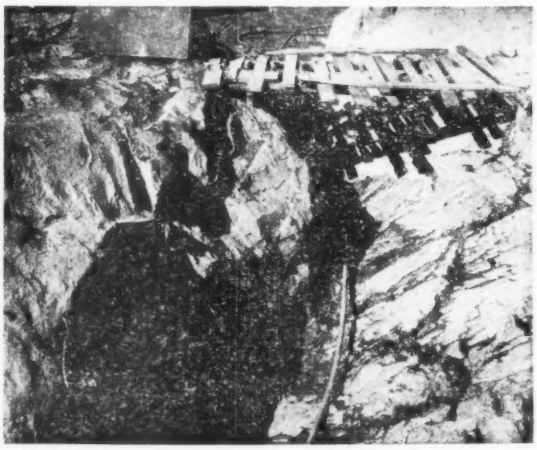
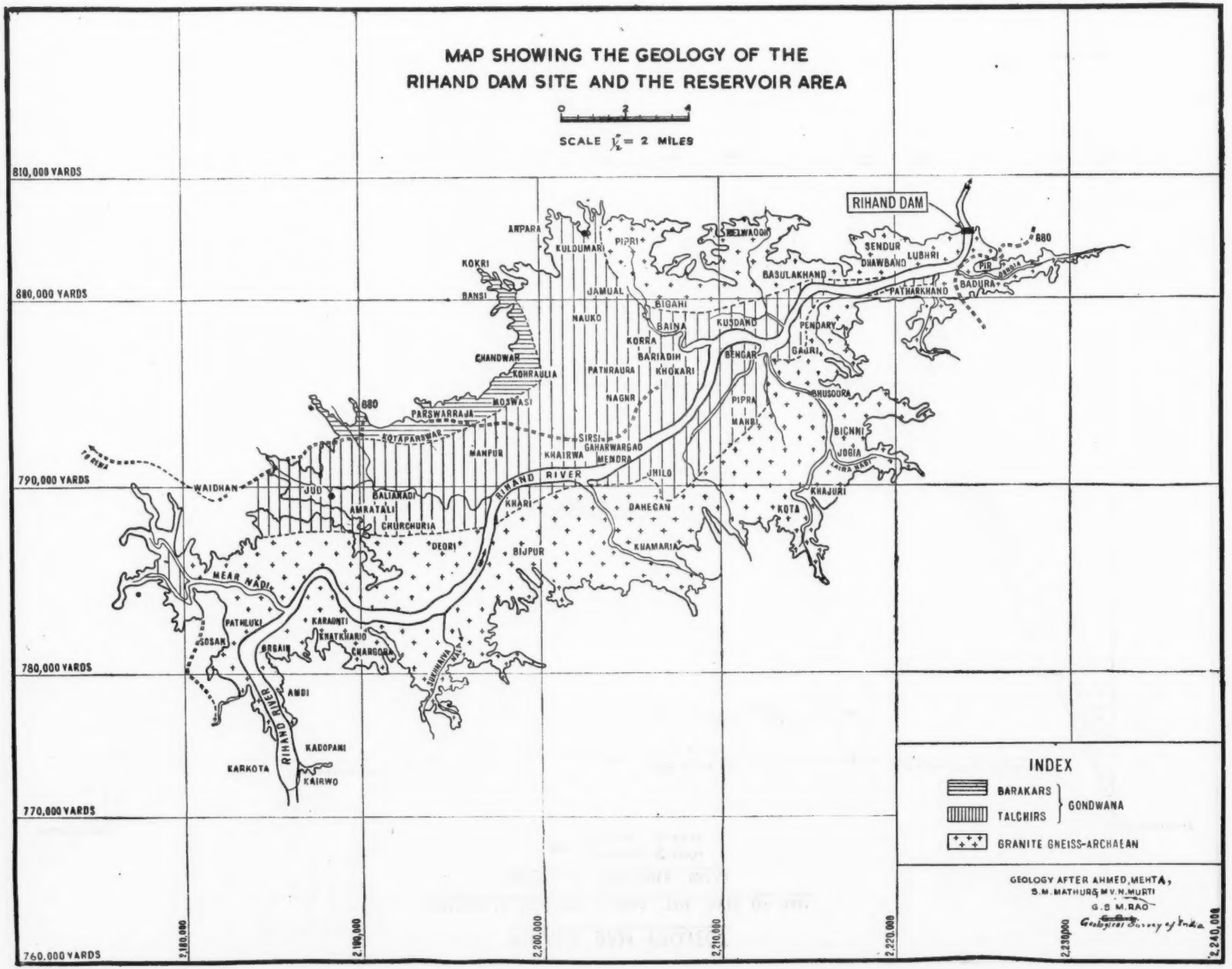


Fig. 1.—Trenching along the shear zone in block No. 28.

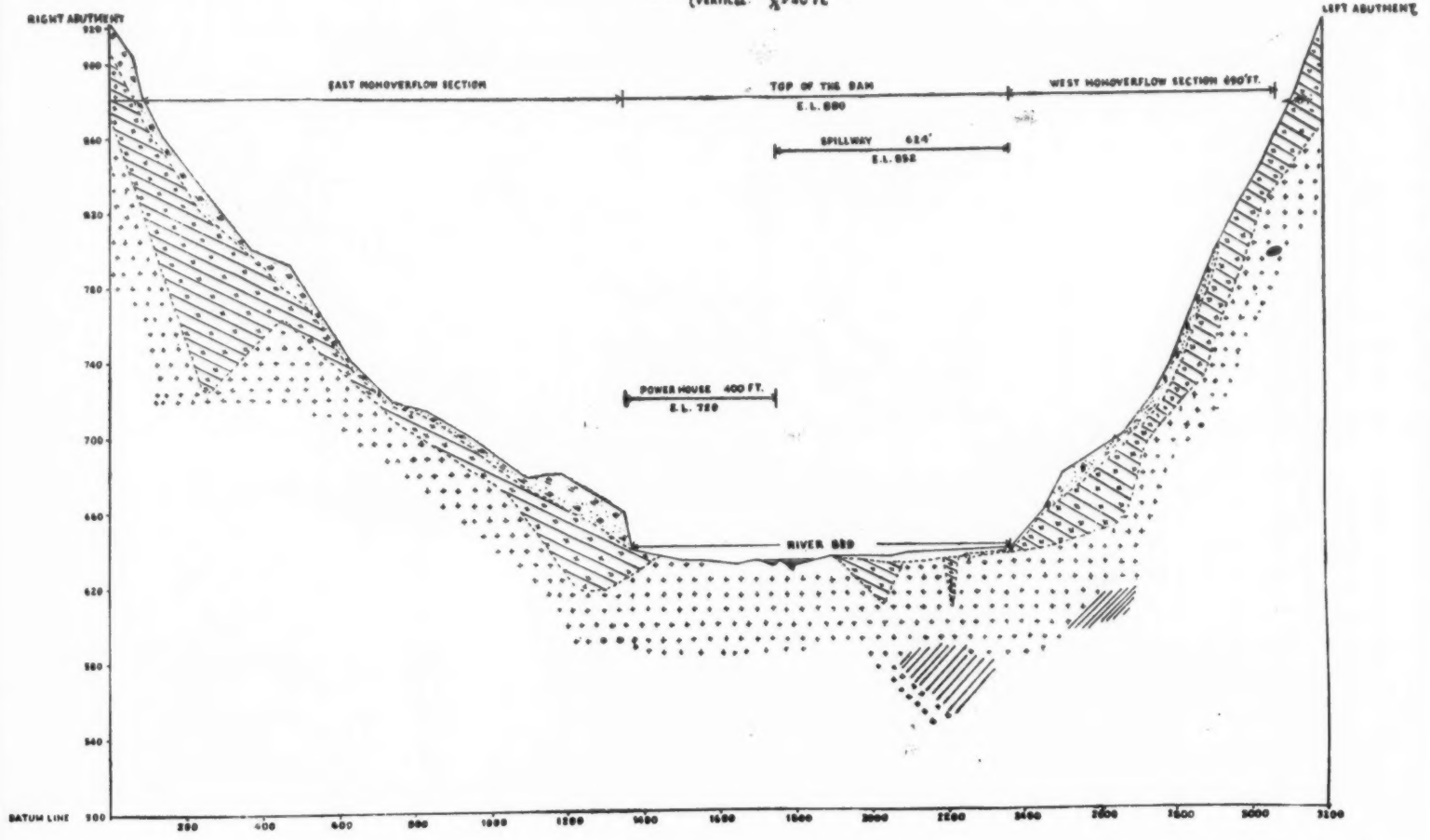

MAP SHOWING THE GEOLOGY OF THE
RIHAND DAM SITE AND THE RESERVOIR AREA
SCALE 1/2" = 2 MILES
810,000 YARDS
800,000 YARDS
790,000 YARDS
780,000 YARDS
770,000 YARDS
760,000 YARDS
2,180,000
2,190,000
2,200,000
2,210,000
2,220,000
2,230,000
2,240,000
RIHAND DAM
AMPARA
KULDUMARI
PIPRI
KOKRI
BANSI
JAMUAL
BIGAHI
NAUKO
BAINA
KUSDANO
KORRA
BARIADIH
CHANDWAR
KOHRAULIA
PATHRAURA
KHOKARI
BENGAR
GAURI
PENDARY
BASULAKHAND
SENDUR
DHAWBAND
LUBHRI
PATHARKHAND
BADURA
PIR
BHUSOURA
NAGNR
PIPRA
MAHRI
BICHHI
JOGIA
PARSWARRAJA
MOSWASI
KOTAPARSWAR
SIRSI
GAHARWARGAD
MENDRA
MANPUR
KHAIRWA
JHILO
WAIDHAN
JUO
BALIANADI
AMRATALI
CHURCHURIA
DEORI
RIHAND RIVER
KHARI
DAHEGAN
KHAMARIA
BIJPUR
KOTA
KHAJURI
MEAR NADI
PATHLUKI
KARAONTI
KHATKHARIO
SOSAR
BROAIN
CHARGORA
AMDI
RIHAND RIVER
KADOPANI
KARKOTA
KAIRWO
INDEX
BARAKARS
TALCHIRS
GONDWANA
GRANITE GNEISS-ARCHAEAN
GEOLOGY AFTER AHMED, MEHTA,
S.M. MATHUR & M.V.N. MURTI
G.S.M. RAO
Geological Survey of India

RIHAND DAM PROJECT

GEOLOGICAL SECTION ALONG THE AXIS OF DAM

(BASED ON BORE HOLE DATA)

SCALE: { HORIZONTAL 1" = 200'ft
VERTICAL 1" = 40'ft.



INDEX

CLAY AND SAND

WEATHERED GRANITE

FRESH GRANITE

INJECTION GNEISS

QUARTZ SCHIST

angles towards the upstream and the downstream (Fig. 1) respectively, were proposed to be grouted and were provided with fillets where necessary.

A high-pressure-grout-curtain, which would extend to a depth of 100 ft. in the river bed and 75 ft. in the abutment, is being provided.

The sheared contact of the granite with the injection-gneiss in the power-house portion was explored and found to die out at R.L. 590 in depth although at the surface it appears to extend upto the quarry located nearly 3 miles to the west of the dam, where also the shearing of the granite and its contact with the injection-gneiss is clearly seen, along with the mineralized zone of pyrite, chalcopyrite and pyrrhotite.

The sheared zones and weathered seams extending below fresh rock are being treated by excavation according to formula depths and back-filling with chilled concrete (Fig. 2). Consolidation grouting is

Fig. 2.—Thin shear zone seen in the downstream wall of the left abutment.

being done along the shear zones. Heavy grout consumption in some of the holes, located along the shear zones in the left abutment, has confirmed the possibility of their effective consolidation by grouting. The geological features of the foundation during the progress of excavation and prior to the concreting of the blocks are being regularly mapped on large scale by the resident geologist to enable the engineers to formulate the treatment of any weak zones prior to concreting, and also for future guidance in case of any necessity for treatment during the operation of the dam.

INVESTIGATIONS FOR CONSTRUCTION MATERIALS

Coarse Aggregate: During June, 1954, after reviewing the availability of construction materials and their relative economics, a high-powered commission, appointed by the Government of India, decided to construct a concrete dam in preference to a stone masonry one. This decision was taken not because of the shortage of rubbles for masonry but because of the other technical and economic aspects.

The Rihand Dam would need about 63.3 million cubic feet of concrete which is supposed to be greater than the quantity needed in the construction of all the seven pyramids of Egypt. In the absence of appreciable gravel deposits near the dam-site, location of a suitable quarry in the granites for about 60 m.c.ft. of aggregate was undertaken with the aid of detailed geological explorations.

Alhough the vicinity of the dam-site abounds in scattered hillocks of granite on the upstream side, a single source which could feed the gigantic crushers with a capacity of 120,000 c.ft. per day had to be located for economic working. The occurrence of injection-gneisses and phyllites on the downstream side of the dam considerably restricted the possibility of locating a course aggregate quarry in the vicinity, without being flooded by the reservoir.

After detailed exploration, the quartzites of Bhoma Pahar and Rakshi Hill on the right bank were found unsuitable because of their being highly crushed, thus giving rise to smaller-sized aggregate on crushing, moreover, secondary silicification of the quartzites might also promote alkali-aggregate reaction.

The granite band of the Chunawa *nala* on the right bank of Rihand, on exploration by drilling, was found to be capped by an overburden of nearly 40 feet on an average. As the quarry would have to be operated from within the reservoir, prevention of flooding during advanced stages of construction would involve expensive protective measures. Hence, the granite band of the Makara hillock situated at R.L. 1250 at about 2½ miles from the dam on the left bank was explored in detail by about 30 drill holes totalling about 3000 running feet. This deposit has been found suitable and is being quarried now.

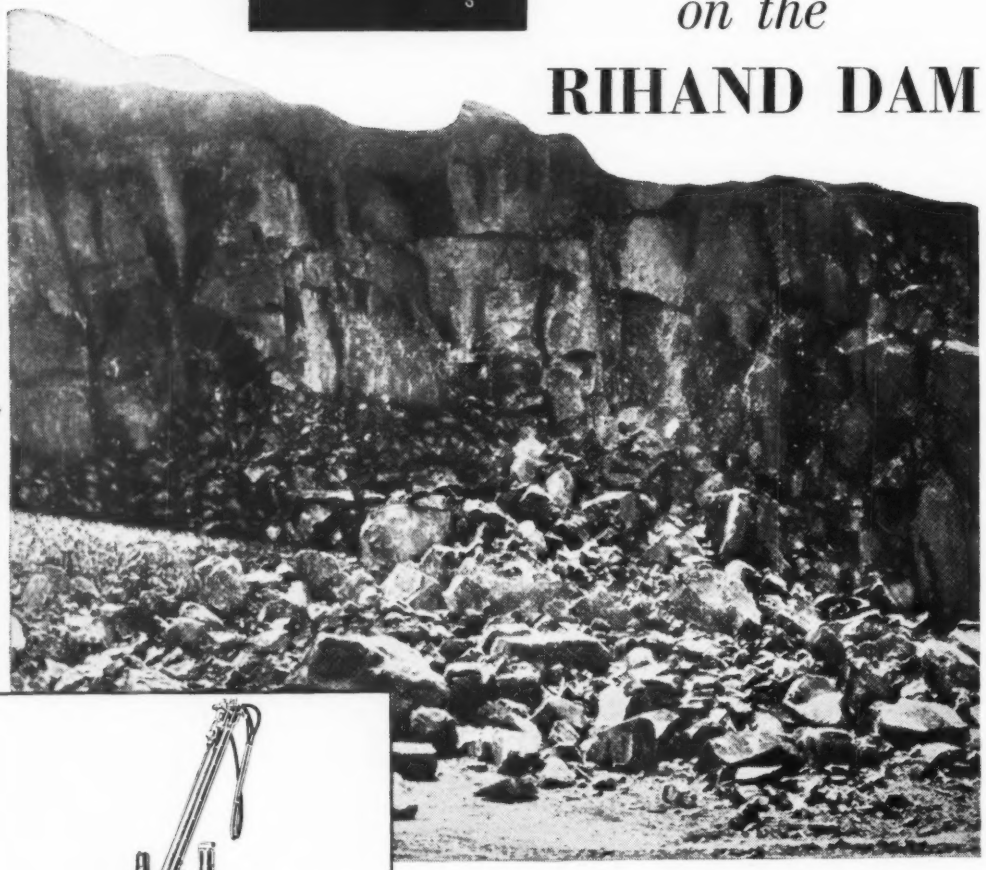
Fine Aggregates: The sand bars in the river bed both upstream and downstream of the dam were examined and the reserves were estimated to be capable of meeting the demand for fine aggregates. The sand with a F.M. of 2.4, which is free from deleterious ingredients, was recommended. But owing to the necessity for blending the various grades and stock piling, the contractors found it more economical to crush the granite for the production of fine aggregate. However, later it was found that mixing of the river sand with crushed granite was inevitable as the manufactured sand was not properly graded.

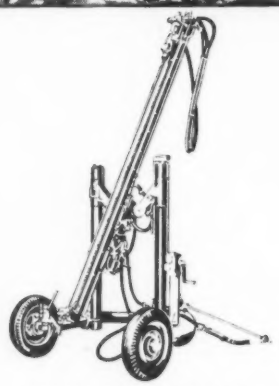
Pozzolanic Material: The Bijawar phyllites and the Barakar clays, occurring in the vicinity of the dam, suspected to be possessing pozzolanic properties, were investigated. As these required calcination and also as the results from the samples were not very encou-

(Continued on page 22)

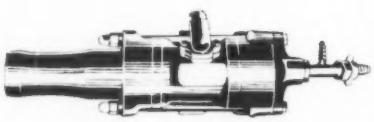
Atlas Copco *at work on the*

RIHAND DAM





WAGON DRILL



ROCK DRILLS

Atlas Copco have supplied the full range of compressed Air Equipment for the Rihand Project such as Air Compressor, Rock Drill, Wagon Drill, Sandvik Coromant Drill Steel, Grinder and other pneumatic fittings.

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VTC-41

General Design and Construction Features of the Rihand Dam

by S. K. JAIN

RIHAND River Project will utilise the potential energy of the water flowing down the Rihand river which is a tributary of river Sone. The construction of a 306 feet high concrete dam has now commenced to achieve the object. The site of the dam lies in Mirzapur district near Pipri about 100 miles from Mirzapur. The river passes through a narrow gorge in a length of about a mile and a half and the particular site in the gorge selected for the construction of the dam is almost ideal for a high concrete dam, having fresh massive granite rock exposed in the river bed free from fissures and faults. Suitable construction materials are also available in close proximity on either bank of the river.

The Rihand river above the dam site drains an area of 5148 square miles with an average rainfall of about 56 inches per annum. The maximum recorded flood is of the order of 4.5 lakh cusecs whilst the minimum flow during the 10 years of record is about 50 cusecs. The average annual run-off at the site works out to about 5.13 million acre feet out of which almost 90% occurs during the monsoon months. All this inflow will be collected in the lake formed by the Rihand dam which will have a capacity of 8.6 million acre feet and an expanse of 180 square miles. The water so collected will be utilized to generate 912 million units of electrical energy every year by the construction of a power house on the downstream of the dam having an ultimate installed capacity of 300,000 kilowatts. The power so made available will be used for industries, for pumping water from tubewells and rivers for irrigation, and for urban electrification in the state of Uttar Pradesh. The supplies of river Sone thus augmented by water released from Rihand reservoir can further be picked up by the Bihar State and utilized for irrigation and power generation.

GEOLOGY OF THE SITE

Extensive geological surveys and sub-surface explorations have been done at the dam site. The type of rock in the river gorge is mainly phyllites, mixed with streaks of gneissose granite, one band of which is 600 ft. wide along the river and is very prominent. Sub-surface explorations of this granite band have revealed no fault planes or objectionable joints or fissures and it, therefore, presents an ideal foundation rock for the dam. No blanket grouting will be necessary by way of treatment of the foundation rock, except in a few locations where joints or seams are encountered.

TOPOGRAPHY

The valley of the river upstream of the gorge in which the dam has been located is very wide and the average slope in the river bed in the vicinity of the dam is only 3.5 feet per mile. The capacity and the surface area of the Rihand reservoir at various elevations have been computed and are shown in figure 1. A lake 180 square miles in area will be formed by 306 feet high dam giving a storage capacity of 8.6 million acre feet which is more than the capacity of the 740 feet high Bhakra dam.

The non-monsoon flow of the river at the dam site diminished to 50 cusecs only and is confined in a very narrow channel and has the minimum water surface elevation at 630. The highest flood level so far recorded is 668.00, at which elevation the water spread at the dam site is 1200 feet. The right bank of the river rises gently above the river level while the left bank rises rather steeply. The length of the dam at the crest will be 3065 feet.

The general contours of the site are shown in figure 2 which indicate that the river hugs the left bank downstream of the dam site, and the spillway, therefore, must preferably be located on this flank of the dam. The noses of the slightly jutting out spurs at the two banks are evidently most suitable to form the abutments of the dam.

HYDROLOGY

As is usual with such projects, the hydrological observations in the catchment of the river commenced when the project was actually envisaged in 1945. We have by now 12 years observation of gauges and discharges of the river from which data the gauge and discharge relationship for the river has been developed. Rainfall records, however, extend much farther back to the year 1903 which have been utilized for making a long term analysis of the available supplies in the Rihand river at the dam site by establishing a relationship between weighted average rainfall in the catchment and the run-off.

Rainfall records of only two stations were available from year 1903 to 1912, three stations from 1913 to 1923, four stations from 1924 to 1931, six stations from 1932 to 1944 and fifteen stations from 1946 to

1949 and five stations from 1949 to 1954 and again only four stations in 1955.

The values of monthly weighted average precipitation over the catchment have been computed for the year 1902 to date on the basis of weightage for each station being obtained by Thiessen's polygons drawn for each combination of station on record from year to year. Incidentally the values of weighted precipitation computed on the basis of any number of stations in a year (2 to 15) have a variation coefficient of 1% only which establishes the usefulness of the Thiessen's polygons.

Weighted precipitation run-off relationship has been established from the observed data of Rihand river over a period of 10 years, by plotting on a log scale, the cumulative figures of (precipitation minus runoff) i.e. 'losses' against cumulative weighted average precipation commencing 1st June, to end of June, July, August, September, October and May of the next calendar year (figure 3) for each hydrological year. The relationship so worked out corresponds to the equation $R = P - 1.17(P)^{0.86}$ where R = Run-off and P = Precipitation i.e. rainfall. By the use of this relationship and having known the values of P since the year 1903, the run-off of Rihand River has been computed for all the years since 1903 to determine the power potential of the river.

Earlier an attempt was made to correlate the hydrology of the Rihand river with the Sone, for which actual run-off records are available at the Sone Canal Headworks (Dehri-on-Sone) about 90 miles downstream of the confluence with the Rihand. This study indicated that runoff of the Rihand river at dam site is equal to 27% of run-off of the Sone river at Dehri-on-Sone for the non-monsoon season and it equals $\frac{R_s \times P_r}{P_s}$ for monsoon season where the suffix 's' denotes the Sone river and the suffix 'r' denotes the Rihand river.

The values of run-off obtained by this earlier method were about 10% higher than those now computed on the basis of observed data of Rihand river itself.

ESTIMATION OF MAXIMUM FLOOD

As already stated, on account of meagre hydrological data it was quite a problem to determine the value of the maximum probable flood to be taken as the design flood and also the value of the maximum flood to be safeguarded against causing failure of the dam.

Evidently a spillway and the dam, designed for the maximum possible flood (i.e the largest flood that can theoretically occur at a given site of the dam over present geologic and climatic era) would eliminate all known chances of failure but the cost of such a provision would be rather excessive. A practical and economical proposition is to design the dam and spillways safe for the maximum probable flood (ie. the largest flood that can reasonably be expected at the site) and to provide complete security (against overtopping of the dam as well as against failure of the dam due to the tensile stresses at the heel of the dam exceeding reasonable safe limits) in the event of occurrence of the maximum possible flood.

The value of peak of maximum flood for Rihand Dam has been calculated in various ways namely (i) by Manning's formula using the data obtained from local physical indication such as flood marks etc. (ii) by the fourteen available empirical formulae, (iii) by unit hydrograph method (iv) by developing an enveloping curve for the peaks of floods throughout the world. The figures of peaks obtained by these methods are:

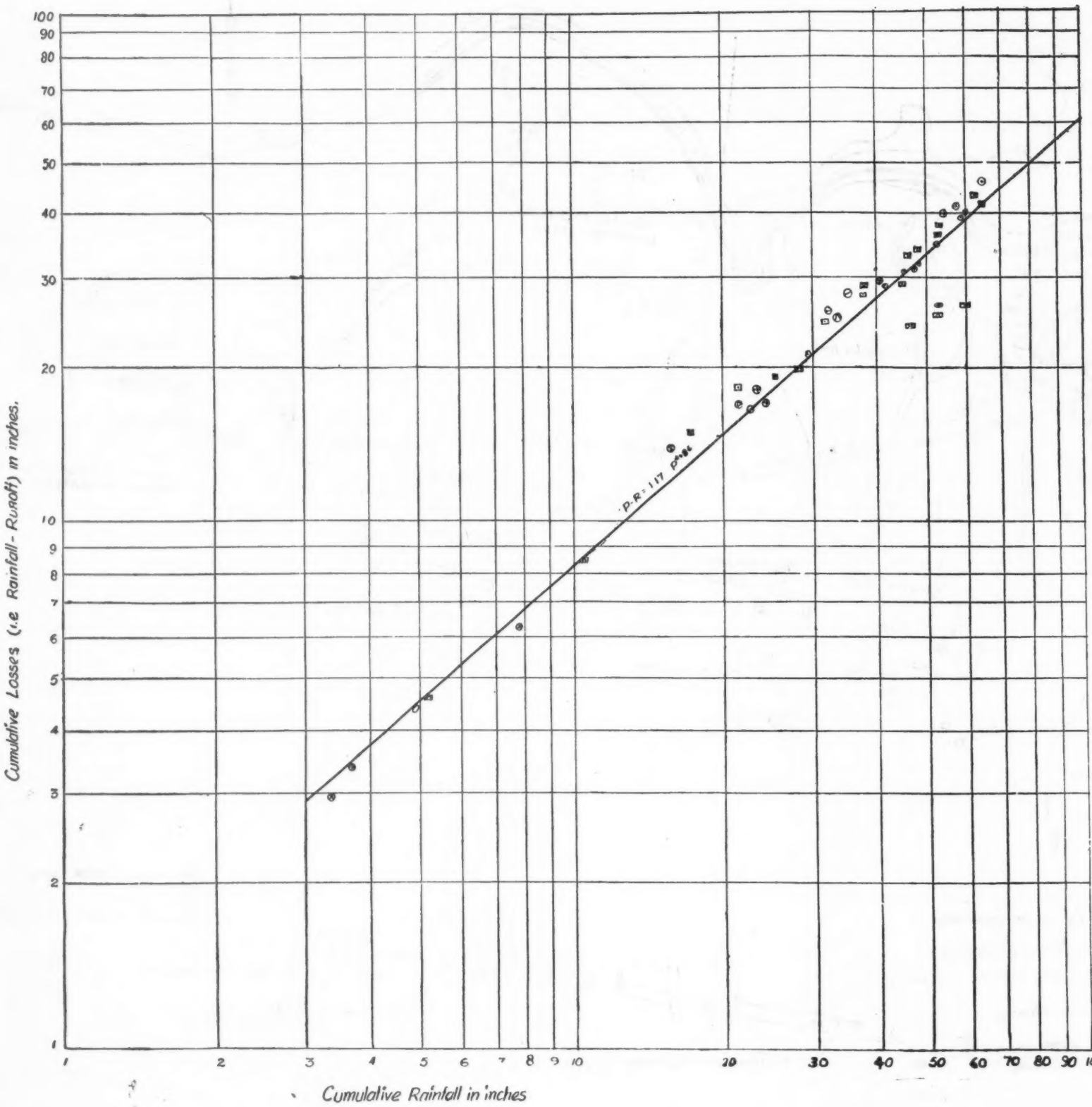
By Manning's formula and local physical observations.	4,60,000	cusecs
By empirical formula Myer's	7,18,000	,,
By empirical formula Dicken's	6,10,000	,,
By Unit hydrograph methods	6,10,000	,,
Regional enveloping flood $Q = 46 \times 100(A)^{0.894}$	$= 7,45,000$	,,
World enveloping flood $Q = \frac{1,31,000 \times A}{(107 + A)^{0.78}}$	$= 8,41,000$	,,

The unit hydrograph method is the most rational and provides for reasonable safety in the assumption of a higher rainfall intensity of 15″ in 24 hours instead of 10″ in which region the Rihand dam site lies according to the rainfall map of India. The peak flood obtained by this method works out to 6,10,000 cusecs which figure has been adopted as the peak of design flood for Rihand dam. It has, however, been ascertained that the world enveloping peak of 8,41,000 cusecs would also be routed with sufficient free board to prevent even wave wash splashes on the top of the dam and without developing tensile stresses in excess of 25 lbs./sq. inch at the heel of the dam. As a matter of fact the rarity of occurrence of maximum possible flood is now recognised every where so much so that for concrete dams a calculated structural damage is allowed during the passage of maximum ever possible flood, provided that the security of the dam is not jeopardized.

The hydrographs for the world enveloping flood with a peak of 8,41,000 cusecs has been developed on the similarity of design flood hydrograph and is expected to be as shown in Figure 4.

TYPE OF DAM

Excellent foundation conditions and the non-availability of good soil for the construction of earth dam, limited the choice of the type of the dam between two alternatives, namely, the gravity or a multiple arch. Design studies, however, indicated that the multiple arch type would have been as costly as the gravity type whereas it would have had two dis-



LEGEND

YEARS	NUMBER OF STATIONS
1945-46	6
1946 47	6
1946 47	15
1947 48	15
1948 49	5
1948 49	15
1949 50	5
1950 51	5
1951 52	5
1952 53	5
1953-54	5
1954 55	4

R = Runoff.
P = Precipitations

IRRIGATION DEPARTMENT, UTTAR PRADESH.
DESIGN AND PLANNING DIRECTORAT
RIHAND DAM

CUMULATIVE RAINFALL, & LOSSES CURVE FOR RIHAND CATCHMENT

DESIGNED :- J. M. Gorga.	SUBTTD. Sd. J. M. Gar
DRAWN :- R.N Agnihotri.	RECMD. Sd. K. M. Mahesh
CHECKED :- Sd. J. M. Garga.	APPVD. Sd S.K. Jain

FIG. 8

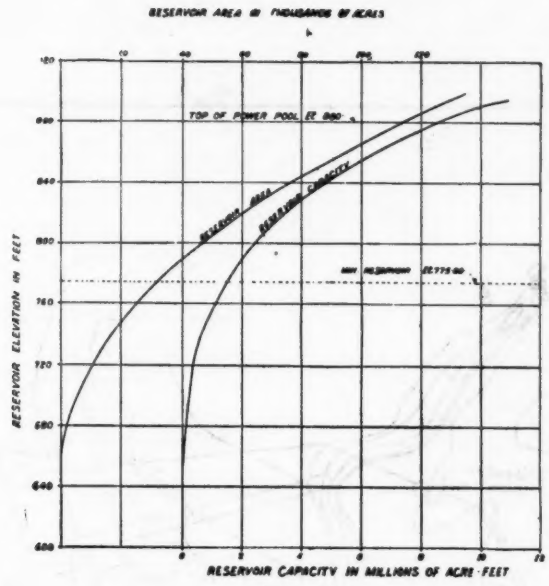

RESERVOIR AREA IN THOUSANDS OF ACRES
TOP OF POWER POOL EL 880
RESERVOIR AREA
RESERVOIR CAPACITY
RESERVOIR ELEVATION IN FEET
RESERVOIR CAPACITY IN MILLIONS OF ACRE-FEET

FIG. 1

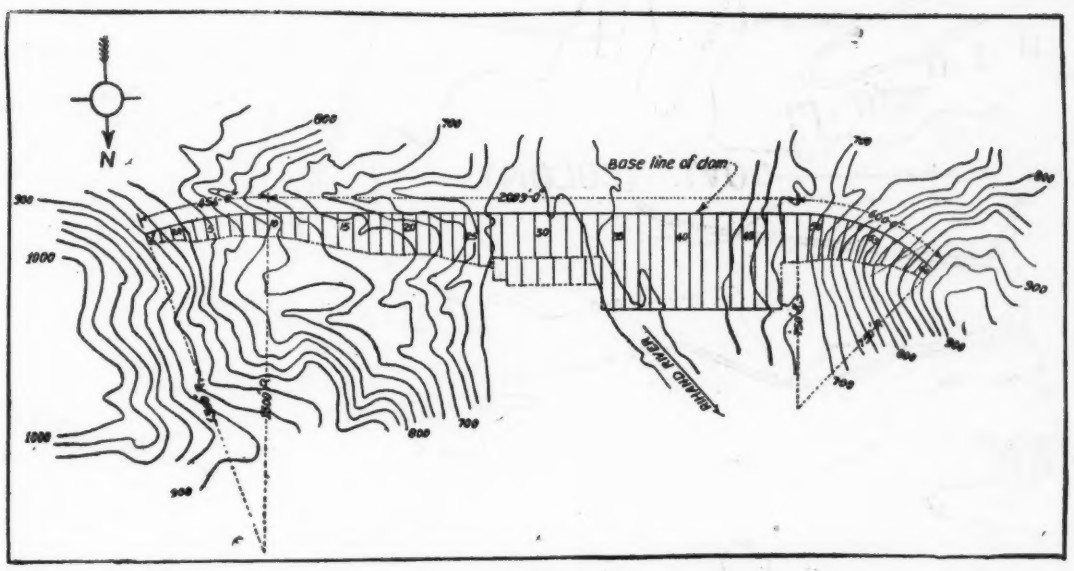


FIG. 2

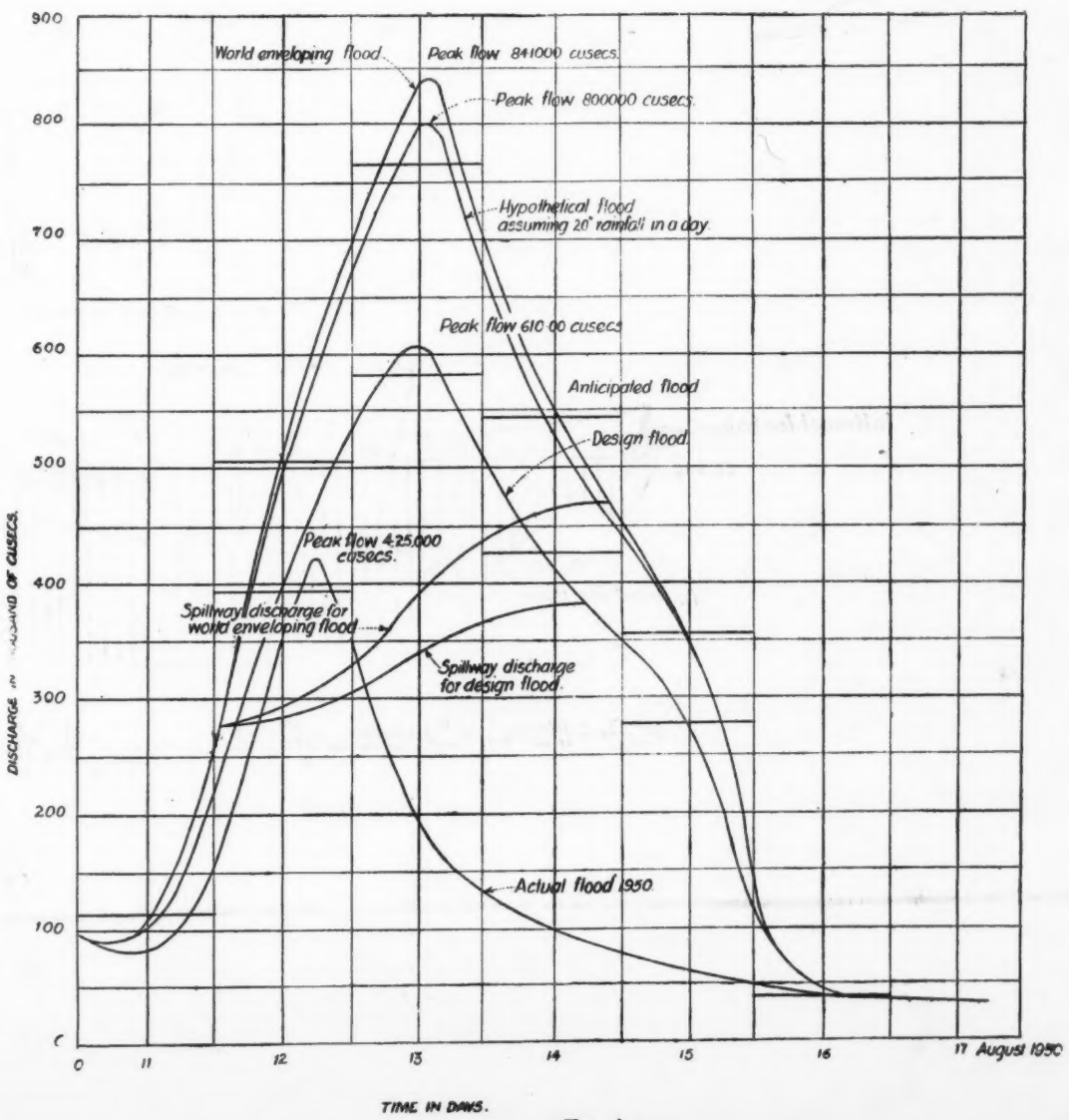

World enveloping flood
Peak flow 841000 cusecs.
Peak flow 800000 cusecs.
Hypothetical flood assuming 20" rainfall in a day.
Peak flow 610 00 cusecs
Anticipated flood
Design flood.
Peak flow 425,000 cusecs.
Spillway discharge for world enveloping flood
Spillway discharge for design flood.
Actual flood 1950
DISCHARGE IN THOUSAND OF CUSECS.
TIME IN DAYS.
17 August 1950

FIG. 4

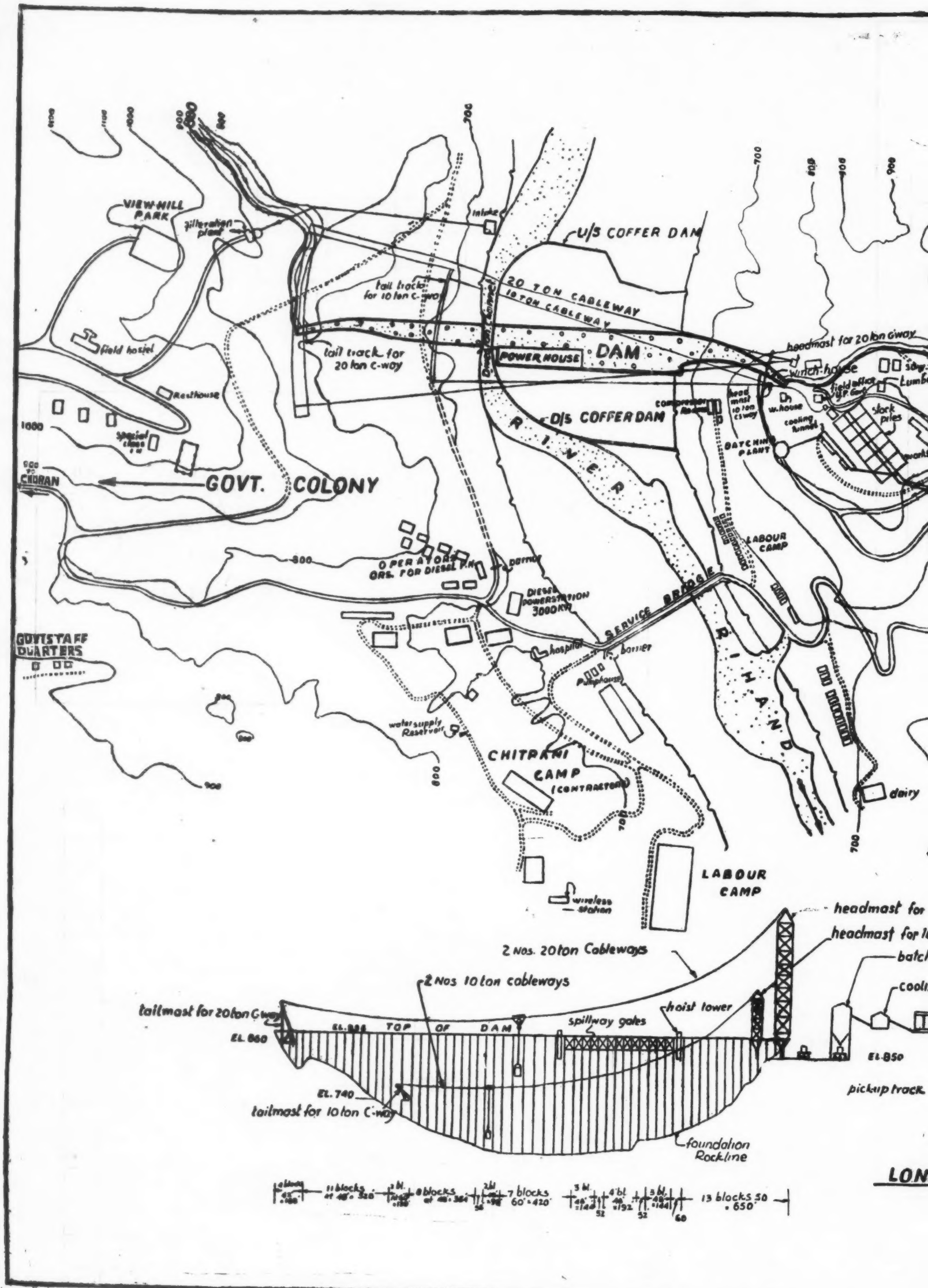
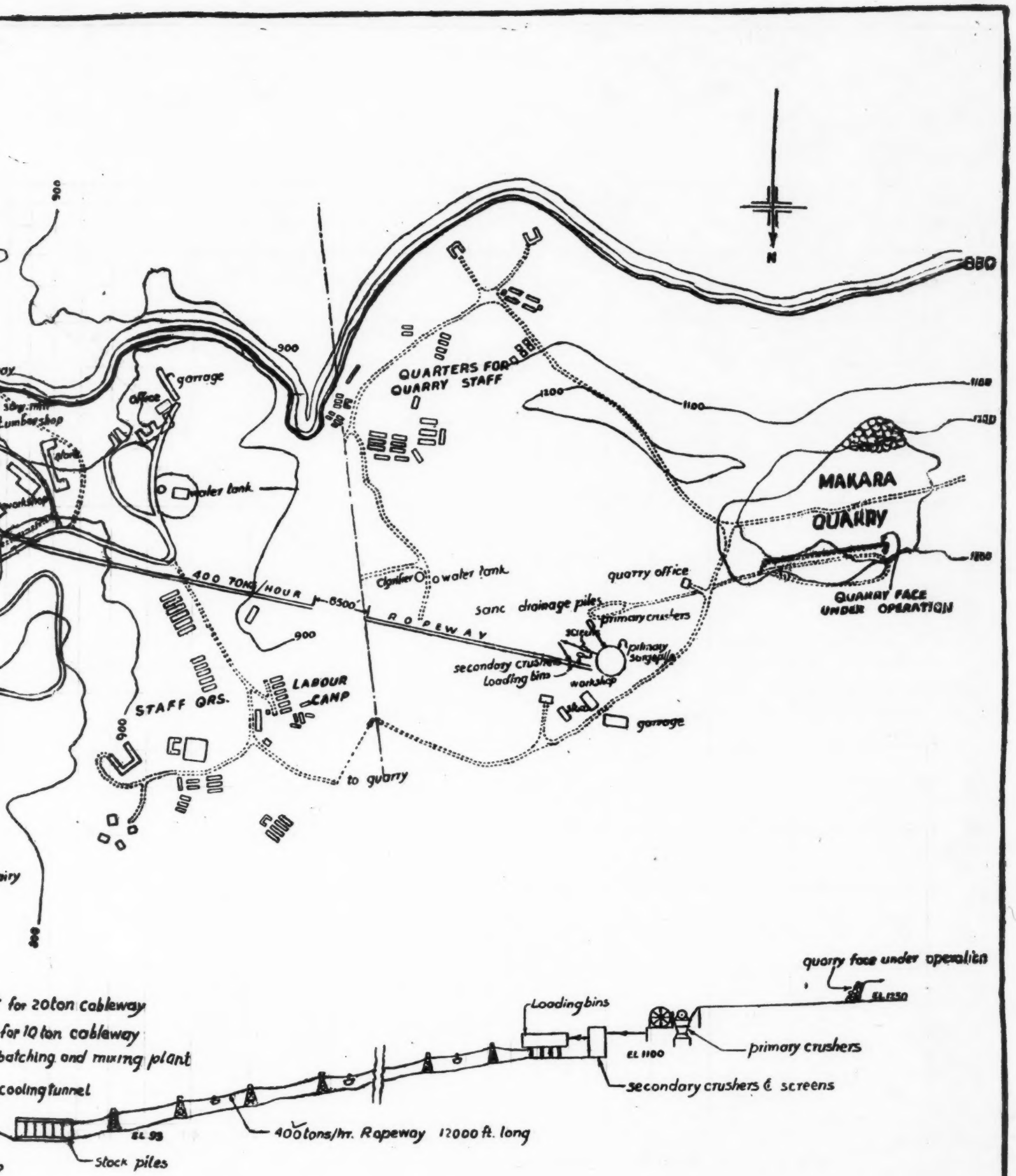

VIEW-HILL PARK
filteration plant
Intake
U/S COFFER DAM
tail tracks for 10 ton C-way
20 TON CABLEWAY
10 TON CABLEWAY
field hostel
tail track for 20 ton C-way
POWER HOUSE
DAM
headmast for 20 ton Cway
winch-house
Rest house
D/S COFFER DAM
RIVER
BATCHING PLANT
GOVT. COLONY
CHORAN
LABOUR CAMP
OPERATORS QRS. FOR DIESEL P.H.
DIESEL POWER STATION 3000 KW
SERVICE BRIDGE
GOVT STAFF QUARTERS
hospital
barrier
water supply Reservoir
CHITRANI CAMP (CONTRACTOR)
dairy
LABOUR CAMP
wireless station
2 Nos. 20 ton Cableways
2 Nos 10 ton cableways
tailmast for 20 ton C-way
hoist tower
spillway gates
EL. 888 TOP OF DAM
EL. 860
EL. 850
EL. 740
tailmast for 10 ton C-way
foundation Rockline
pickup truck
11 blocks at 48' = 528'
8 blocks at 48' = 384'
7 blocks 60' = 420'
13 blocks 50' = 650'

FIG. 6



ONGITUDINAL SECTION

RIHAND DAM

CONSTRUCTION PLANT LAYOUT

Scale 1:500 feet

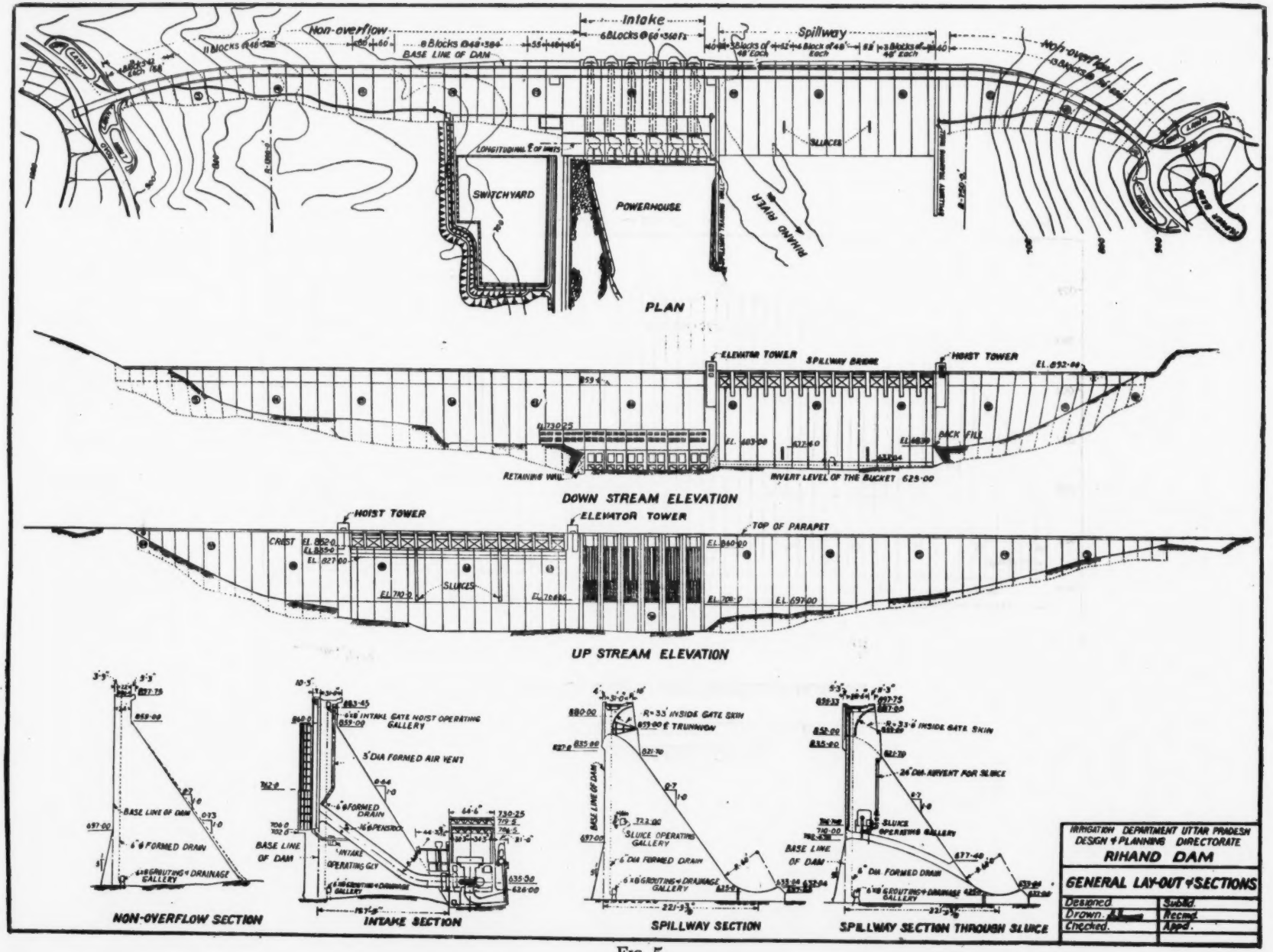

Non-overflow
Intake
Spillway
Non-overflow
BASE LINE OF DAM
SLUICES
SWITCHYARD
POWERHOUSE
RIHAND RIVER
PLAN
ELEVATOR TOWER
SPILLWAY BRIDGE
HOIST TOWER
EL. 652·00
BACK FILL
RETAINING WALL
INVERT LEVEL OF THE BUCKET 625·00
DOWN STREAM ELEVATION
HOIST TOWER
ELEVATOR TOWER
TOP OF PARAPET
CREST EL. 852·0
SLUICES
UP STREAM ELEVATION
BASE LINE OF DAM
6'ø FORMED DRAIN
6'ø GROUTING & DRAINAGE GALLERY
NON-OVERFLOW SECTION
INTAKE GATE HOIST OPERATING GALLERY
5'DIA FORMED AIR VENT
INTAKE OPERATING GLY
INTAKE SECTION
R=33' INSIDE GATE SKIN
SLUICE OPERATING GALLERY
SPILLWAY SECTION
24'DIA AIRVENT FOR SLUICE
SPILLWAY SECTION THROUGH SLUICE
IRRIGATION DEPARTMENT UTTAR PRADESH
DESIGN & PLANNING DIRECTORATE
RIHAND DAM
GENERAL LAY-OUT & SECTIONS
Designed
Drawn
Checked
Submd.
Recmd.
Appd.

FIG. 5

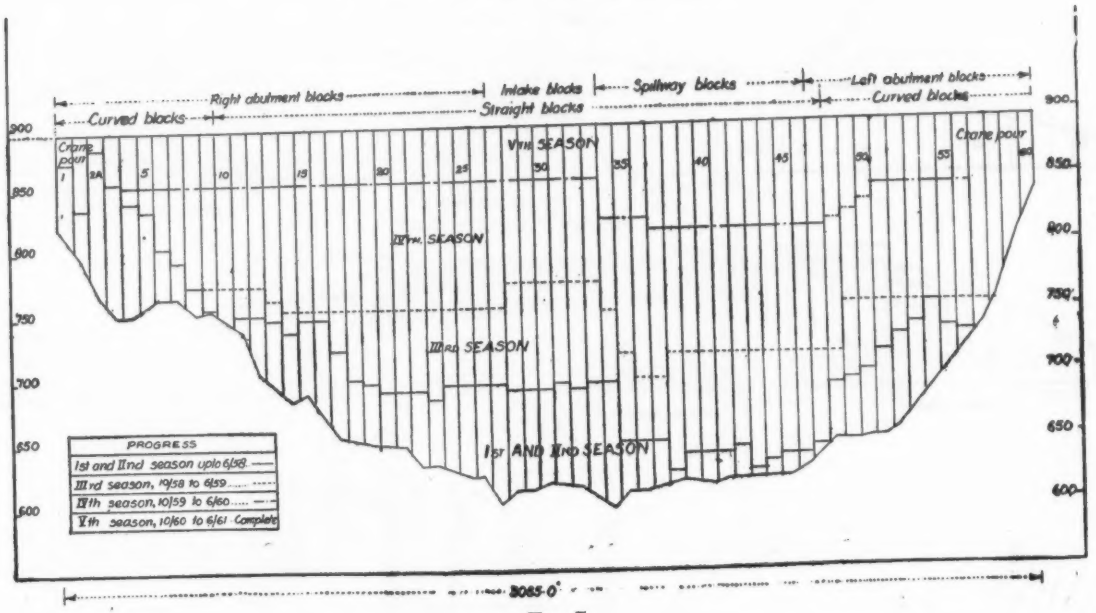

RIHAND DAM
LONGITUDINAL PROFILE
CONSTRUCTION PROGRAMME AND PROGRESS
Right abutment blocks
Intake blocks
Spillway blocks
Left abutment blocks
Curved blocks
Straight blocks
Curved blocks
Crane pour
Crane pour
V TH SEASON
IV TH SEASON
III RD SEASON
I ST AND II ND SEASON
PROGRESS
1st and IInd season upto 6/58
IIIrd season, 10/58 to 6/59
IVth season, 10/59 to 6/60
Vth season, 10/60 to 6/61 Complete
900
850
800
750
700
650
600
3085-0"

FIG. 7

tinct disadvantages, firstly that it would not have been able to carry a roadway on its top and secondly, that it would not have permitted an overflow spillway which would have, therefore, been located on a saddle on left abutment requiring deep excavation and excessive expenditure. Gravity type of dam was, therefore, the obvious choice. There was, however, a controversy of concrete versus masonry but overall economics and safety considerations, including expediency in construction and availability of suitable material and cement nearby, ultimately led to the decision in favour of concrete structure instead of masonry dam.

HEIGHT OF THE DAM

The height of the dam depends on six factors:—

(i) Geological features and foundations.
(ii) Safety.
(iii) Economics.
(iv) The precipitation and run-off available from the catchment.
(v) Topographical features and elevations of saddles in the rim of reservoir.
(iv) Elevation of minerals or mines or other useful natural or artificial assets in the reservoir area.

Whereas the geological features of the foundation and safety of the dam provide practically no limitation for the height of the Rihand dam, the remaining four considerations led to the definite conclusion that the normal maximum reservoir level at the dam should be 880 which elevation of power pool would give the best percentage return on the capital outlay, would fully utilize the available run-off, would need no saddle dams and would not submerge corrundum mines which exist near elevation 890.

The free board above the normal maximum reservoir level to the top of the dam should be such that maximum ever flood can be routed through the reservoir without overtopping. Usually the free board is kept as 5% of the height of the maximum section of the dam and incidentally this figure of 5% = 14.4 feet also works out to be sufficient for the Rihand dam from all considerations. The road level of Rihand dam shall thus be at elevation 894.5 while parapets shall be provided at elevation 897.75 which would give a clear free board of 6.95 feet above the maximum reservoir elevation of 890.8 which may be attained if at all, during the world enveloping flood. The maximum height of the dam shall be 306 feet, as the deepest foundation level is 588.5.

POWER POTENTIAL

The entire storage between the normal full reservoir level 880 and the dead storage level 775 is considered as live storage which works out to 7.28 million acre-feet. The capacity of the reservoir upto dead storage elevation 775 is 1.32 million acre feet which is expected to get silted up in 130 years.

The continuous power output as computed in detail by trial and error, on the basis of hydrological data with due allowance for evaporation at 5.5 feet annually works out to 105,000 kilowatts, over a cycle of 50 years, except for short periods of dry cycle, when power will either have to be rationed or supplemented by other means. The operational hydrograph of the reservoir for generation of 105,000 kilowatts continuous over a cycle of 50 years indicated that the reservoir level will remain generally above elevation 832, except in a few years when it may go down to elevation 815, which corresponds to the critical head of the turbines.

The initial installed capacity of Rihand Dam Power House shall be five units of 50,000 kilowatts each with 70,000 H.P. turbines rated at a head of 225 feet (corresponding to the reservoir elevation of 857). The maximum output of turbines at heads higher than the rated head will be limited to 10% overload to safeguard against excessive rise of temperature in generators. The maximum output of a unit with reservoir elevation at 815 shall be 42,500 kilowatts and with reservoir elevation 832, it would be 50,000 kilowatts so that normally four units will meet the full peak demand, of 1,05,000 kilowatts average at 55% load factor, leaving one unit as spare; but in dry years, the fifth unit will also have to be run to meet the peak load.

There is a proposal to construct a 70′ high dam on the Rihand river at Obra approximately 20 miles downstream of the Rihand dam. Power output from Obra power Plant would be approximately 32,500 kilowatts. Detailed studies for running the Rihand Dam Power Plant in conjunction with the Obra plant and for interlinking it with other power grids have indicated the necessity of future installation of a sixth generating unit so that Rihand Dam Power Plant, having the advantage of storage, could take the peak load of other stations. It has, therefore, been decided to make provision for the sixth set in the power house and to initially complete all the underwater works for the same including the intake structure and the penstocks which shall be initially closed with a hemispherical bulk head gate.

GENERAL LAYOUT OF THE DAM AND APPURTENANT WORKS

The base line of Dam (the line in plan of the vertical upstream face of the dam) has been so aligned that the entire base area of the dam will be founded on solid granite. The alignment of the dam will be straight in the central 2009 feet length, while the ends are curved in lengths of 600 feet and 456 feet having radii of 750 feet and 1300 feet on the left and right flanks respectively. There are no earthern dykes at the ends and the end blocks are laid against steep rock sides. The dam has been divided into 61 blocks, serially numbered from the right end, the 3rd block being called block 2A due to its having been introduced at a late stage. The last block is therefore, numbered 60. The length of the blocks varies from 42 to 60 feet.

The transverse block joints will be plain contraction joints and these shall not be grouted, since grouting would not appreciably help in reducing the stresses in the dam section by transfer of load to abutments. Metal water stops and asphalt seals have been provided at the joint to inhibit the seepage of water into the joint.

The most economical and suitable arrangement of various components of the dam and its appurtenances has been found to be as below:—

(a) Block nos. 1 to 34 and 47 to 60 comprise the non-overflow portion of the dam.

(b) An overflow spillway has been located on block nos. 34 to 47 with piers in the centre of the blocks and training walls at either end in block nos. 34 and 47. There are 13 bays 40 feet long in the spillway which will have crest at elevation 852. The capacity of the spillway will be sufficient to safely route the Design Flood and it has also been ascertained that the dam will not be over-topped even if the world enveloping flood occurs with the reservoir already full upto elevation 880. The position of the spillway also suits topography and the river course downstream of the dam.

(c) The power house has been located at the toe of block no. 28 to 33, with an erection bay at the toe of block no. 27.

(d) The transformers have been located on a deck between the dam and the power house.

(e) Switchyard has been located on the right bank just downstream of the power house.

Parking areas would be provided at both the ends of the dam for facility of parking vehicle and to provide suitable approach to the road on the top of dam. A small parking place would also be available near the power station. The layout and sections of the dam are given in Figure no. 5.

NON-OVERFLOW SECTION OF THE DAM

The top width of the section has been kept at the economical figure of 14% of height of dam i.e. 24 feet which together with a cantilever projection on the downstream allows for the provisions of a 22 feet wide roadway and 3 ft. 9 inch and 5 ft. 3 inch wide pavements on the two sides. The upstream face is vertical from top of dam to elevation 697.00 below which it slopes at 1 in 5. The downstream face is vertical below the top cantilever upto elevation 859.00, below which it slopes at 0.7 to 1.0 on the east non-overflow section and 0.73 to 1.0 on the west non-overflow section. The slope in the intake blocks 28 to 33 is 0.64 to 1.00, as reduction in concrete has been possible on account of intake structure on the upstream.

OVERFLOW SECTION OF THE DAM

The spillway section has its crest at elevation 852.00. It is slightly wider than the non-overflow section for reasons of stability and hydraulic efficiency. The upstream and downstream faces have the same slopes as the non-overflow section on the east.

The maximum water surface elevation for routing the design flood would be 886, which gives a design head on the crest of 34 feet. The profile of the spillway surface has been designed corresponding to the lower surface of a jet issuing from a sharp crest under a head of 34 feet. The spillway surface conforms downstream of the crest to the equation $(X/H_e)^{1.85} = 2(Y/H_e)$ with origin at the crest, X axis being horizontal, positive downstream and Y axis vertical positive downwards. The profile upstream of the crest is a quadrant of a circle with radius $0.282\ H_e$.

The adopted profile of spillway may develop negative pressures at certain locations on the crest under partial openings of the crest gates and for reservoir elevations above 886 (i.e. when H_e is greater than 34 feet.) Such negative pressures, however, would be rare and are not likely to exceed $0.1\ H_e$ i.e. 3.4 feet head of water which figure is insignificant, and does not warrant any arrangement for aeration.

The profile of the spillway upstream of the crest is a quadrant of a circle of $0.282\ H_e$ feet radius and projects out four feet from the upstream face of the dam to improve flow conditions.

The discharge over the spillway crest will be regulated by tainter gates of $40' \times 28'$ in size, each of which in fully lowered position will have its top 2.34 feet above the full reservoir elevation 880. Enough clearance has been provided under the spillway bridge to permit full travel of these tainter gates. The hoists for the tainter gates will be located on piers under the spillway bridge.

BUCKET AND SIDE WALLS

A 60 feet radius and 85 degrees sweep bucket with invert at elevation 625 has been provided to dissipate the energy of the high velocity water flowing down the spillway. Model test carried out at the Irrigation Research Institute, Roorkee, indicated that the bucket has a roller action upto a discharge of 20,000 cusecs if evenly distributted over all the thirteen bays, beyond which it behaves as ski jump bucket. At the maximum probable discharge of 3,83,000 cusecs on the spillway, the ski rises to elevation 706.00 and the normal tail water level is attained at a distance of approximately 445 feet from the lip of the bucket. The model results further indicate that the ski has no tendency for side flow and the retaining wall between the power house and spillway with top at elevation 683.00 is not overtopped. The rock downstream of the bucket is massive and hard and no deep erosion is likely to occur.

To minimize the turbulence in the tailrace and to avoid the deposit of sand in it due to backflow from the spillway, the retaining wall between the power house and the spillway is being extended normal to the base line upto 125 feet downstream of the draft-tube. The retaining wall on the west end of the spillway is continued to only 50 feet downstream of the bucket since erosion of this bank is not indicated on the models.

STRESSES IN THE DAM

The stability analysis of the dam has been based on the assumption that each block behaves independently of its neighbours, the behaviour of the section being that of vertical cantilever rising from the rock foundation. Sections of unit width at the base line were analysed for safety against sliding and overturning. Stresses caused by various loads such as normal and flooded reservoir conditions, uplift pressures, wave pressures and earthquake corresponding to 0.1g acceleration have been verified to be within safe limits.

Measuring instruments such as strain meters, stress meters, thermometers, joint meters, plumb bob and uplift pressure pipes and gauges have been embedded and installed within the dam in block nos. 26 and 41 so that the actual behaviour of the dam as compared to theoretical computations could be studied after the construction of the dam.

PENSTOCKS AND INTAKE STRUCUTURE

Each turbine in the powerhouse shall be supplied water by a 16 feet dia. penstock inside the dam. The opening shall be lined with steel plates suitably designed for the hydrostatic head and water hammer, except in the transition from rectangular opening at the inlet to circular penstock section. Emergency gates shall be provided on the upstream face of dam to close the penstock inlet and thus save the turbine from attaining the runway speed due to sudden off-load in the event of jamming of the turbine wicket gates. A 36″ diameter air vent is provided downstream of the intake gate to reduce the negative pressure downstream of the gate during closure. Upstream of the emergency gate, provision has been made to put in stoplogs to isolate the intake gate and the penstock for maintenance. Trashracks are located upstream of the stoplogs in the cage type structure which projects upstream beyond the body of the dam. The bottom of the intake structure has been kept at elevation 706.00 so as not to obstruct the natural net of flow lines approaching the penstock entrace. The top of trashrack structure is kept at elevation 860.00 which level is expected to be attained practically every year when it would be possible to take out the racks for maintenance.

SLUICES

The water stored in the reservoir of Rihand dam will be exclusively utilized for power generation and consequently the regulation will primarily depend on power demand from time to time. No irrigation outlets or undersluices are really necessary. It has, however been decided to provide the sluices $4' \times 9'$ each, in blocks nos. 38 and 43 with centre line of the inlet, at elevation 710, so that assured supply of 5,000 cusecs could be released from the reservoir, even in the rare emergency of copmlete shutdown of Rihand Power Station, to feed the Obra Dam Power Plant which may come up in future, 20 miles downstream of the Rihand dam. Operation of sluices to regulate supplies or for routing of floods would not be necessary but the possibility of utilization of these openings for construction and for washing out silt has been kept in view in deciding the elevation of these sluices.

The size of the conduits would be $4' \times 9'$ as the gates having height equal to approximately twice the width work out to be most economical. The outlet conduits would have suitable transitions a the inlet ends, conforming to the equations: $\frac{X^2}{D^2} + \frac{Y^2}{(D/3)^2} = 1$ for top and bottom; and $\frac{X^2}{B^2} + \frac{Y^2}{(B/3)^2} = 1$ for sides.

The alignment of the bottom of the conduits would conform to the equation of trajectory for the jet issuing with reservoir elevation at 886. The trajectory for the bottom of the conduits would be suitably inclined at the inlet (by 5 degrees with the horizontal) so that the bottom of the conduits at the exit end becomes tangential to the spillway bucket.

The maximum velocity through the sluices would be 100 feet per second and an airvent would be provided in order to minimize cavitation near the gate slots. The size of the airvent would be 30 inches as obtained by the formula: $\frac{QA}{QW} = 0.04 \frac{(V-1)^{0.85}}{gd}$ assuming maximum permissible velocity of air through the vents of 150 feet per second and the critical demand being at 80% gate openings. A 12″ diameter airvent would also be provided at the exit end of the sluice to supply air in the air pocket formed between the sluice and the spillway discharge. Another 12″ diameter airvent is located downstream of the emergency gate.

OPENINGS INSIDE THE DAM

(a) A drainage and grouting gallery running all along the length of the dam is provided at a minimum height of 5 feet above the foundation, rising along the foundation rock in steps of 7″ rise and 10″ tread. The gallery is connected to power station drainage gallery by a cross gallery in block 34 and all seepage into the dam openings would thus be carried through the power house drainage gallery into a sump provided in the erection bay from where it will

be pumped out. The foundation gallery is connected to the outside of dam in block 2 and 59, through doglegged staircase and adits, and is connected to the penstock operating gallery by a flight of steps in blocks 22 and 23.

(b) The penstock operating gallery is located at elevation 683.67 and is approachable at its two ends from donwstream of the dam by adits in blocks 23 and 34. The gallery provides approach to the penstock filling line valves which are located in chambers upstream and above the gallery, in blocks 28 to 33.

(c) The sluice operating gallery at elevation 722.00 extending from block 34 to 47, provides access to the sluice gate hoists which are located in chambers in blocks 38 and 43.

(d) A gallery has been provided with floor at elevation 883.45 extending from block 28 to 34 to provide access to the motors for the penstock gate hoists, located in six chambers in the side walls of the intake structure.

(e) All the four galleries are connected to the top of the dam by a passenger elevator and doglegged staircase in block 34, a tower on the downstream of roadway being provided for access from the top of the dam. This tower shall have provision of utilities for the visitors.

(f) A shaft would connect the top of dam to the sluice operating gallery and the foundation gallery in block 47. The shaft would be provided mainly for the carriage of sluice gate leaf, and would be used for carriage of grouting machines, cement and other materials to the foundation gallery. The access on top of the dam would be provided through a tower located downstream of roadway and utilities would also be provided for public use in this tower. Necessary blockouts will be left for the provision of a freight elevator in the shaft at a later stage.

(g) Two shafts 2′—6″ in diameter have been provided in block 34, one connected to the reservoir and the other to the tailrace for recording the levels of water. The reservoir gauge well extends from elevation 891.00 to 719.00 and the tail water gauge well from elevation 683.00 to 620.00.

POWER STATION

The main power station building, to house the six turbines and the erection bay, is 420 feet long and 85½ feet wide. It is divided into seven independent units 60 feet long each, by contraction and expansion joints. The power house superstructure is being made in structural steel for expediency in construction. Downstream of the power station, a regulator is being provided with divide walls extending from the draft tube to the regulator. It would thus be possible to isolate any of the draft tubes by putting in planks in the regulator bays, if it is not possible to lower the draft tube gates due to collection of debris in the guides. The regulator in addition helps to avoid the existence of turbulence downstream of the draft tubes when the spillway is overflowing. It will also maintain the minimum tail water level in the early stage when only one machine may be running at half the load with full reservoir level.

A reinforced cement concrete framed gallery structure is provided on the dam from block 28 to 34, just upstream of the power station, having floors at the same elevation as provided in the power station. This structure will have auxiliaries, stores and cables. Transformers will be located on the uppermost floor of this structure at elevation 683.00. The control room shall also be located in a reinforced cement concrete framed building at the toe of blocks 25 to 27. The control building will be connected to the gallery structure through a staircase and lift. A cable tunnel will carry the control cables from the control room to the switchyard.

CEMENT CONCRETE MIX

The mass concrete is designed for an average 28 days strength of 2,500 psi and a design strength of 2,000 psi which must be exceeded by 90% of the specimens.

The concrete in five feet thick mass on upstream and downstream faces of the dam is designed for 28 days strength of 4,000 psi. Extra rich and vacuum processed concrete comprising ¾ inch maximum size aggregate and having 28 days strength of 4,000 psi shall be used in top 1 foot thickness for the following locations where pitting due to cavitation is likely to occur and the velocity would be more than 40 feet per second:

(a) Inner surface of the outlet sluices.
(b) Spillway crest above El. 805.65
(c) Spillway bucket below El.681.56.

The average water requirement in mass concrete containing 6 inches maximum size aggregate has been found to be 195 lbs. per cubic yard of concrete with an air entrainment at $3 \pm 0.5\%$ in 1½ inch concrete mix. Air is being entrained with a view to save cement and a cheap indigenous air entraining agent has been developed at the Rihand Dam Laboratories for this purpose. The water cement ratio by weight is fixed as 0.65 ± 0.02. The cement content per cubic yard of mass concrete is 287 lbs. The slump of concrete so designed is usually 1″.

Fly ash from Bokaro Power Station and locally available fire clay have been found to possess satisfactory pozzolanic activity. Arrangements are being made to start using fly ash as pozzolan in cement concrete. Fire clay will be used as pozzolan as soon as crushing plant is installed at the Dam site to pulverise the fire clay, available at 20 miles from the dam site.

Calculations for thermal characteristics of concrete indicate that for the placing schedule of 5 feet lifts at intervals of 72 hours, the concrete tempera-

ture at placement should not exceed 60°F. This temperature is attained by precooling of aggregates and by mixing chilled water to the concrete.

CONSTRUCTION PLANT

Natural aggregate deposits being not available, the production of aggregate has been planned by crushing of rock. Granite rock, suitable for concrete aggregate, is available in sufficient quantity on the left bank of the river at 2½ miles from the site. The quarrying shall be carried out in 90 ft. depth in an area of 560,000 sq. ft. Crushing of blasted rock and its separation into various size groups is being done close to the quarry and an elaborate crushing and screening plant has been set up for the purpose. The smaller size aggregate is passed through Kubit Breakers to obtain a cubical shape of the aggregate. Sand is also crushed from rock but deficiency in the required quantity is made good by mixing river sand. All sand is passed through a classifier to washout the fines.

Aggregates in six size groups 6″-3″, 3″-1½″, 1½″-¾″, ¾″-3/8″ 3/8″-3/16″ and -3/16″ are transported from the quarry to the stock piles near the dam site by two ropeways each of 200 tons per hour capacity. The stock piles have a capacity of about 5,000 tons of each aggregate. Material is drawn out of the piles on an underground belt conveyor located in a tunnel and transferred on to another belt and shuttle conveyor which carries it to one end of the cooling tunnel, where bins are provided for each type of aggregate. These bins feed six different belts 300 ft. long placed inside an insulated cooling tunnel. As the material is transported on the belts, the coarse aggregates 6″-3/8″ are cooled by spraying chilled water over the belts, while the finer aggregates are cooled by blowing chilled air on the enclosed belts. Coarse aggregate is thus cooled to about 40°F and the fine aggregate to 60°F. The total capacity of refrigeration plant is 990 tons.

The cooled aggregate is collected into separate insulated bins at the other end of the cooling tunnels, and is then carried on an insulated belt conveyor to the batching plant bins. Weigh batching and mixing of concrete is done in a batching plant having four 4-cu.yd. Koehring mixers and Winget Control Panel. The mixers are capable of mixing a batch in 2½ to 3 minutes, revolving at 12 r.p.m. The batching plant can handle twelve different mixes at a time by the use of mix selector knob. The mixed batch is discharged on to a common hopper which delivers the concrete to 4 to 8 c.yd. buckets brought under the hopper on trailors pulled by 50 H.P. locomotives.

The placement of concrete is being done by two 'ten ton' and two 'twenty ton' cableways having spans of 2100 ft. and 3050 ft. respectively. The bigger cableway have the longest span so far used in the world. The travel speed of the bucket is 1500 ft. per minute and the tail tower can move at 100 ft. per minute.

Cement is transported in bulk from Churk Cement Factory, 50 miles away, by 18 cement carriers each of 16 tons capacity. Two silos of 500 tons capacity each at Churk and two 1000 tons silos at the batching plant provide about a week's storage. The cement carriers unload the cement in a pit from where it is raised by horizontal screw conveyor and vertical elevator to the storage silos. The same elevator is again used for elevating cement from the silos to the batching plant bin.

The layout of the plant is shown in Figure 6.

PROGRAMME AND PROGRESS

Preliminary construction work on the project was commenced in 1948. A year later, however, the work had to be stopped due to urgent necessity of transferring all available personnel, materials and funds to the short term food production schemes. A decision to resume the construction was taken in October 1951, and an estimate for the project work was sanctioned in 1952. A master plan for the execution of the project was prepared in 1953-54. Meanwhile financial arrangements were finalised with the Government of India. An operational agreement between the Government of India and the Government of United States was signed on March 29, 1954 to meet the foreign expenditure of the project. A Control Board with the Chief Minister of the State as Chairman was formed in the same year to act as supreme authority for the project. Tenders for the construction of the main dam and powerhouse were received in January 1955 and the work was given to M/s Hindustan Construction Co. Ltd., Bombay.

The preparation of working drawings for the dam and powerhouse and specification drawings for the equipment was started in February, 1955, when a Design and Planning Directorate was established. Specifications and drawings for the control equipment such as gates and hoist have since been completed and tenders for the supply of most of the equipment have been received and are being scrutinized. Detailed designs of various features of the dam and power station are also nearing completion. Construction drawings are being prepared and issued along with the progress of work.

The total concrete in the dam and appurtenances is estimated to be 6,00,00,000 cu. ft. The project is scheduled to be completed by June, 1961 but the dam will come upto a sufficient height to store water upto El 820 during monsoon of 1960, so that one of the 50,000 kW. generating units may be commissioned in October, 1960, to partially meet the long felt demand of power. The remaining four machines will be put into commission in quick succession by the end of 1961.

Work on excavation of foundation for the dam was started in April, 1955 and arrangement for procurement and installation was almost complete by June, 1957. Placement of concrete was commenced in April 1957 by the two 10 ton cableways mainly

for the construction of tail tracks across the dam. About 11,50,000 cu. ft. of concrete was poured during the period ending August, 1957 when the operations were closed for the monsoon. The work was recommenced after the monsoons on 15th October, 1957 and a total of 1.85 crore cu. ft. of concrete has been placed upto 1st December 1958. The concreting programme for the dam is shown in Figure no. 7.

REHABILITATION PROGRAMME

The area to be submerged by the Rihand lake comprises 40,000 acres of forest land and 46,000 acres of cultivated land in Uttar Pradesh, the corresponding area in Madhya Pradesh being 15,000 acres each. The rehabilitation programme involves shifting of 108 villages having 40,000 persons in Uttar Pradesh and 44 villages with a population of 15,000 in Madhya Pradesh.

Full compensation is being paid for the land and other immovable property, while colonies are being encouraged close to the boundry of the reservoir where free land is being given and other facilities such as drinking water, approach roads etc., are also being provided. No land revenue shall be charged from the settlers in these colonies for the first three years. The land alloted to each family is 5 acres or two thirds of his original possession.

BENEFITS

The Rihand Dam Project is primarily a power project the main object being the industrial development of the state by providing power for the establishment of major and minor and cottage industries. The mineral resources available in the area round about the Rihand Dam site and the agricultural and forest products available from the eastern and south eastern regions of the State offer bright prospects for the establishment of electro-chemical, metallurgical and other basic industries such as manufacture of cement, chemicals, fertilizers, aluminium, porcelain and paper etc., with the help of the large block of power to be provided by the Rihand project at an attractive rate.

Service area of the Rihand project is very intensively populated and people are consequently economically backward as agricultural operations alone cannot provide employment for more than a fraction of the total population. Cottage industries helped by electrical power are essential for bringing up the present low standard of living of the people. The skill and craftmanship already exists and power from Rihand project will enable the industries to flourish. There is large demand of power both for small scale consumers goods and feeder industries as well as for the heavy and cottage industries.

The scheme will further benefit the State of Bihar to a great extent as the reservoir will provide a minimum flow of 50,000 cusecs of water throughout the year which may be utilized for the extension of Sone Canal irrigation system in that State.

The capital cost of the project including transmission lines of 132 and 66 kV lines will be 45.26 crores and it is estimated that 912 million units (kilowatt hours) of electrical energy will be annually available from the project at a cost of 2.32 naya paise per unit at the generating station.

(Continued from page 34)

STRUCTURAL AND HYDRAULIC DESIGN OF DAM AND ITS APPURTENANCES

dules are given for all reinforcement to facilitate bending and cutting of bars to the exact shape. In all about 500 drawings would be required for the dam and its appurtenances. Lift drawings are not prepared are are neither considered necessary at site.

DESIGN FACILITIES

It has been the first opportunity for us to work on a major dam project and though we had enough experience on small dams, the problems of structural design of Rihand dam and its appurtenances were new to us. The team of engineers working on the design has been drawn from the State service and comprised mostly of fresh graduates. Their enthusiasm for the work has been the chief reason for our success. We did not have the advantage of training in the design offices of outside countries, more advanced in the dam building operations, nor we had foreign experts for guidance. I have to gratefully acknowledge the help given to us by the C.W.P.C. and Bhakra design personnel with whom I had the privilege of discussing my difficulties. Though such visits were very few, the information in respect of the practices adopted on the design of Bhakra and other dams was of great assistance to us.

(Continued from page 13)

GEOLOGY OF THE RIHAND DAM PROJECT

raging, it was found that it would be more economical to use fly-ash from the Bokaro Thermal Plant as a pozzolan.

The author wishes to express his gratitude to Shri A. C. Mitra, I.S.E., Chief Engineer, U.P., Irrigation Department, for the facilities provided to him in the furtherance of his duties and the keen interest he always evinced in the progress and results of the geological investigations. He is thankful to the Project engineers for the kind cooperation extended to him.

The author expresses his grateful thanks to Dr. P. K. Ghosh and Shri P. C. Hazra, Superintending Geologists, Geological Survey of India, for the valuable guidance he has had from them during the course of investigations. He is also indebted to the latter for the guidance in writing this article and editing the same.

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THE HINDUSTAN CONSTRUCTION CO. LTD.

The Hindustan Construction Company Limited., founded by that Captain of Industry Seth Walchand Hirachand, started its operation 34 years ago. It was launched in 1926 as a subsidiary of the Tata [now Premier] Construction Co. Ltd., with an initial paid-up capital of Rs. 1 lakh.

The authorised Capital of the Company to-day is Rupees one crore and subscribed and paid-up capital is Rs. 61 lakhs.

The salaries and wages bill of the Company in the year 1930 amounted to Rs. 2.74 lakhs and during the last financial year it reached the figure of over Rs. 2 crores. Similarly this steep curve has also been maintained in the annual turnover of business. From a monthly turnover of Rs. 80,000/- two decades back to a figure of Rs. one crore [Rs. ten million] is an achievement of which the Company could feel amply proud.

The jobs in hand and under execution on this day amount to around Rs. 37 crores [about 80 million dollars] comprising, as they do, constructional activities of various kinds, viz., Bridges, Dams, Power Houses, Tunnels, Public Health Engineering Jobs, Industrial Buildings, Civil Works in connection with Steel Plants etc., etc., for the Central and State Governments.

To-day the Company has in its employ nearly 400 qualified engineers and over 2500 technical supervisory staff, besides a very large number of skilled technical personnel. Most of the engineers are qualified university men and quite a few of them have also had training in foreign countries.

Amongst the top personnel are men connected with technical education, professional engineering institutions, labour organisations and engineering industries and occupy high positions of responsibility in social and industrial spheres. The top core, so to say, is a reservoir of long standing experience of construction works and technical skill and ability of a high order.

With the activities spread over a very wide field, the Company is fully aware of the high demands of modern machinery. It has, at present, machinery of all kinds, viz., earth moving, heavy concreting machinery for construction of dams, bridges, roads etc., worth nearly Rs. 40 million working on the various jobs. Great attention is paid to their maintenance and replacement of obsolete types in order to take the utmost advantages of latest innovations and improvements in this field.

During the last five years the Company's activities have reached a high mark in various fields of construction. Amongst its current jobs mention may be made of the 18 crores Rihand Dam Project envisaging the construction of an all concrete 296′ high dam for the Uttar Pradesh Government. The magnitude of this job can be understood when it is known that the total concrete that would be poured is over 22,50,000 c.yds. and 8.6 million acre feet of water would be impounded in the reservoir that would be formed.

Besides the Company is also handling the civil engineering works in connection with the Bhilai Steel Project, the construction of the underground power house for the Koyna Project [Bombay State] [the first of its kind in Maithon in DVC was recently completed by it] the Brahmaputra Bridge in Assam etc., which are very important projects requiring quality performance and workmanship.

The Ganga Bridge at Patna which was recently opened was also constructed by this Company —being the largest bridge structure in this country where foundations are carried to 170′ below the bed.

Verily the Hindustan Construction Company is a unique engineering and construction organisation in this country who have played a considerable part in the development of Construction Industry in India and whose contributions to the projects under the Plans are no mean.

With the resources, experience, knowledge, technical know-how, high talented staff and technical personnel this Company hopes to play a still greater part in the engineering and construction field in the years to come. Once again the organisation has exploded the myth that our nationals can only be second best. It has proved by its performances that it stands par with any organisation in any country.

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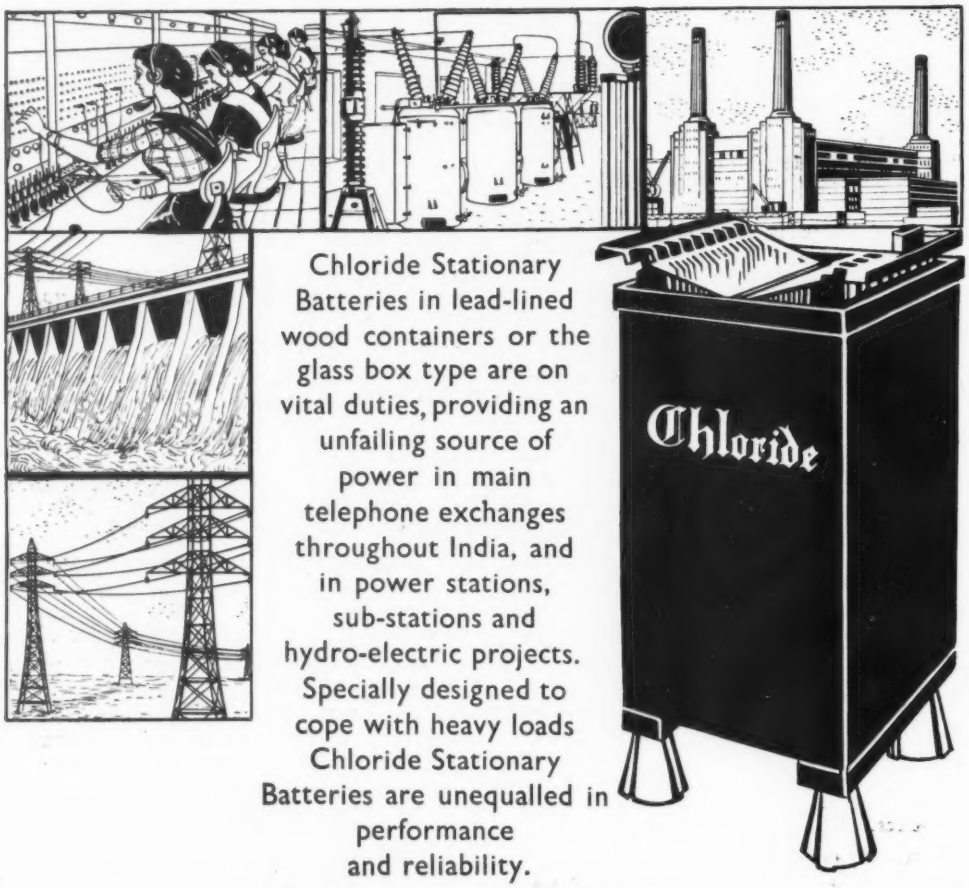

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Structural and Hydraulic Design of Dam and its Appurtenances

By JAGDISH NARAIN

THE Rihand Dam adds another bright star to the galaxy of dams scattered all over the country. Possessed with excellent foundations and reservoir storage, the engineers and geologists have unanimously acclaimed it to be an ideal scheme, and the construction of a gravity dam as most opportune. The design of a gravity dam is often considered to be within the domain of school boys and to a certain extent it is true when one compares it to the design work involved in arch dams or multiple arch or buttress dams. A generalisation such as this would however be an injustice to the gravity dam designers who may be faced with sufficiently complicated problems such as in very high dams or where ideal conditions do not exist, or where special features are required. Devoid of any complication worth mentioning or special features requiring intricate design procedures, the Rihand Dam may be defined as an ideal book structure. The only redeeming feature in Rihand designs is the maximum economy consonant with safety and full use having been made of the bounties of nature, so profusely offered at the site. The fundamental design criteria for the structural and hydraulic design and various assumptions made in the design analysis are briefly given hereinafter.

DIVERSION ARRANGEMENT

The very wide valley at the dam site allowed of only one possibility for river diversion, that of working in part width and allowing the river to flow in other part. The choice had to be made between high coffer-dams at a high cost for isolating the working area throughout the year or arranging the isolation of the work area during the non-monsoon months only. Hydrological observations indicated that the maximum discharge in the non-monsoon months did not exceed 10,000 cusecs in most of the years on record. This discharge could be effectively diverted from the work area at a much lower cost and enable uninterrupted work for nine months in a year. The diversion programme was thus based on the work area being isolated for the non-monsoon period only and allowing its flooding during monsoons. Very satisfactory working of the diversion arrangement in the last two years has justified the decision.

A major part of the river bed made available for excavation and concreting operations by constructing low masonry coffer dams, aligned along the rock exposures in the river bed (Fig. 1.). Such an alignment for coffer dam solved the foundation problems for the coffer dam as well as resulted in a tight water seal. The coffer dam presented a zig-zag shape, but was most economical. To allow for good flow conditions during overflow in monsoons, the downstream coffer dam was given an ogee profile. The upstream coffer dam had a trapezoidal profile and it was considered unnecessary to give this an ogee profile as it would most of the time overflow under submerged conditions. The river flow was confined in a narrow width on the right bank by constructing a diversion channel with masonary side walls, high enough as not to be over-topped during the maximum non-monsoon flow. The area to be occupied by the channel and the two side walls in the dam foundations was excavated to sound rock prior to building the masonry walls.

The diversion channel carried the non-monsoon flow till May 1958 during which time concreting of the dam within the coffer dam was carried out to sufficiently high levels and the entire foundation rocks was practically covered with concrete. A diversion conduit was made in block 37 while the adjacent block no. 38 was kept at low level. The side walls of the diversion channel and its walls were raised, water being diverted through the diversion conduit in block no. 37 and on top of block no. 38. Flow during monsoon 1958 was confined to the spillway blocks 35 to 47, which were at low level, and through the diversion conduit in block no. 37.

The non-monsoon flow is now confined to the diversion conduit and all the dam blocks are available for concreting to higher levels. During this concreting season the spillway blocks shall be carried upto a lower level as compared to the adjacent non-overflow blocks so as to avoid overtopping of the non-over-flow blocks during monsoon 1959, when water will pass over the low blocks and through the diversion conduit. Arrangement will be made to lower a bulkhead gate at the inlet to the diversion conduit before monsoon 1960, to enable filling of reservoir to sufficient high level to work one turbine in October 1960.

To safeguard against the possibility of river supply not going down so low as to give the blocks free of water soon after monsoon, arrangement will be made to install water tight shuttering on the low blocks.

Post monsoon inflow in 1960 shall pass through the permanent conduits in blocks 38 and 43, till the turbines are not put into operation.

FOUNDATION TREATMENT

Keying of a gravity dam inside the foundation is required to obtain additional safety against sliding. Sliding is not a problem at Rihand since the foundation rock is a solid monolithic mass without well-defined joints or bedding planes and it does not slope downstream except in isolated areas. Solid rocks are overlaid practically all along the dam, except in the river bed, by sufficient depth of weathered rock. Even the weathered rock could be considered good enough for the foundation of a gravity dam of the magnitude of Rihand and it has often been pointed out to us that what we have removed was much better than what they have considered satisfactory at Bhakra, Grand Coulee or Boulder. The weathered rock had crushing strength greater than the strength of concrete in dam at most of the places. The removal of such weathered rock pockets has been guided by a possible further deterioration in course of time. Weathered rock enclosed by sound granite and not likely to be subjected to the deteriorating action of percolating water, was not removed. Excavation has not been done inside sound rock except a little by wedging and barring, in locations where sufficient depth of weathered rock was removed. At other places, the anchorage inside sound rock has been planned to be 5 feet where the dam is more than 200 feet high and about 2½ feet for the portion of dam of lesser height.

Permissible slopes along the base line of dam as well as cross slope normal to base line are often required to be specificed for guiding the excavation. While no limits were placed on the slope along the base line, the cross slope was fixed on the requirement that the shear friction factor along the foundation line should not exceed the safe limits. For the deepest portion of the dam, the permissible maximum slope worked out to be 1 in 20.

The few seams of weathered rock in the foundation were specified to be treated by excavating to a depth equal to twice the width of seam and backfilling with cooled concrete. Attempts to grout the weathered seams did not meet with any success. Sharp corners were specified to be rounded off, as far as possible, to reduce the possibilities of cracking under thermal variations.

FOUNDATION GROUTING

The rocks being of the Bijawar Series and providing almost one solid mass all along the dam foundadation, no consolidation grouting was considered necessary. The foundations, however, presented a deep seam in block 27 and 28 (Fig. 2) which extended into the power house, running at the base line of dam. This seam was grouted by drilling pattern holes as shown in the sketch the seam being excavated fairly deep to remove all badly weathered material and to obtain enough concrete depth for effective arching. The number of cement bags taken by each grout hole is indicated in the figure.

Further programme of grouting of the dam fountion is to create a grout curtain by drilling through pipes rising from the foundation rock to the foundation gallery and embedded in concrete. (Fig. 3) These pipes are of four inches diameter and are provided to save this depth of drilling through concrete. The spacing of grout pipes has been kept five feet. However, grouting shall be done by split spacing method and all the holes may not be drilled. The depth of drill holes has been fixed to be 100 ft. for portion of dam above 200 feet in height and 75 ft. for portion of dam below 200 ft. The grouting of holes will be done by stage grouting, the maximum pressure used being 200 psi. All the grout holes will be drilled at an angle to the vertical necessary inclination having been provided in the embedded pipes on the basis that the bottom of the hole may not be upstream of the heel of dam. This would avoid leaking of the grout into the reservoir bottom and would place the grout curtain close to the heel of the dam. The water cement ratio for grout will be fixed in accordance with the water loss in the particular hole, which would give an idea of the tightness of the hole. The pressure grouting shall be carried out only when the dam has been raised to a sufficient height so that the foundation is adequately loaded and no upheaval occurs under grout pressure.

Grouting through holes inclined towards downstream with inlet at heel, is proposed in blocks 27 and 28 only, to treat the seam of weathered rock mentioned earlier.

DESIGN OF DAM SECTION

The Rihand Dam has three distinct zones—the non-overflow, the overflow and the intake i.e. the portion having penstocks. The overflow and the intake portions are straight in plan while the non-overflow portion is partly straight in plan and partly curved. The profile of section in these reaches has been fixed by obtaining stability and safety under the applicable loads. For the purpose of design analysis, the loads have been divided into two categories:—

(i) the normal loads under the application of which the permissible stresses will not be exceeded, and

(ii) the abnormal loads for which the stresses may be allowed to exceed the permissible stresses, though still remaining below higher limits termed as the emergency values.

The permissible and emergency values are indicated under each requirement for stability.

The normal loads considered are:

(1) Dead loads

(2) Reservoir at Elevation 880.00

(3) Tail water at Elevation 630.00
(4) Uplift acting on the base.

The abnormal loads have been taken as:—

(i) Higher reservoir conditions during floods i.e. reservoir elevation 886.00 and 890.80 corresponding to the routing of the design flood and the maximum possible flood, corresponding tail water levels being 679.00 and 680.60

(ii) Wave wash due to high wind velocities, and

(iii) Seismic forces.

The canyon for Rihand Dam necessitates a 3065 ft. long dam for a 300 ft. height. The dam profile thus does not allow a substantive reduction by transferring the load on to the abutments by beam action and this is the reason why the block joints have not been grouted. Water stop provided between the joints shall hardly transfer any stress from one block to the other and the twist action due to curvature of the dam at the ends and unequal heights of two adjacent monoliths are assumed insignificant.

Stability analysis of the dam has thus been based on considering a slice of the dam one foot wide at the base line and contained between two vertical planes normal to the base line. The sides of the slice remain parallel in the straight length of the dam and the width of the section remains one foot throughout; while in the curved lengths the width of the slice reduces towards the toe. This is the reason why flatter slope is required on the left curved portion. The stability of the blocks having penstocks has been considered by taking the block as a whole.

While working out the stresses in the dam slice, the openings are neglected as these are considered to have a local effect only and according to St. Venants principle, the openings would not affect the general distribution.

UNIT WEIGHT OF CONCRETE

The weight of concrete has been taken as 150 lbs. per cu.ft. The laboratory determinations indicate a value of 154 lbs. per cu.ft. for the unit weight but the higher figure was not adopted in the design computations to account for possible variations in the field. The dam would therefore have somewhat greater stability than indicated in the calculations.

WATER LOADS

The water pressure due to reservoir and tailwater have been calculated by hydrostatics, dividing the forces into the vertical and horizontal components, where the dam is sloping, the vertical component being equal to the weight of water on the face. (Fig. 4). During higher reservoir levels, the spillway gates shall be raised and the force of water on the section is taken as the triangle of pressure with the top portion of the triangle above the level of the crest removed.

The unit weight of water has been taken as 62.4 lbs. per cu.ft. Calculations indicated that it would take approximately 130 years before silting takes place upto elevation 706.00 just upstream of dam. The Rihand water silt content is not more than 300 parts per million as found by actual measurement of silt during monsoon. The tailwater has been considered active on the non-overflow section of the dam.

WATER FORCES

The height of waves in the reservoir has been worked out by Molitor's formulae $h=0.17\sqrt{VF}$, where F is the fetch of the reservoir in miles and V, the velocity in miles per hour. The fetch of Rihand Dam lake is 30 miles and the sustained wind velocity expected in the reservoir area is 30 miles per hour. Both the dam and the lake are aligned East West approximately, and the main lake is separated from the dam by a narrow extending normally upstream of the dam for about half a mile (Fig. 5) The dam is rather shaded from the direct waves in the lake and only reflected waves would reach the dam. The height of waves 'h_w' reaching the dam has been taken to be the same as worked out for the case of a dam having direct waves from the reservoir since the hill side near the dam are steep and reflected waves will be strong. The height of wave works out to 5.10 ft. The rise in water surface when the wave breaks against the dam would be to a height $1.5h_w$ i.e. 7.6 feet and pressure due to this rise would be given by $125h_w^2$ i.e. 3260 lbs. per ft. of dam. More elaborate calculations for wave forces have been taken in the design of radial gates, though the simple procedure is considered adequate for the stability of the dam.

SEISMIC FORCES

The effect of earthquake has been considered in horizontal direction only. The dam site, according to the map of seismic zones in India, lies in the region of very moderate damage. However, a value of g/10 has been taken for the seismic acceleration as recommended by the Geological Survey of India. The seismic force on the body of the dam is worked out by multiplying the mass of dam by the seismic acceleration. The seismic effect on water pressure is calculated by Westergaard theory.

UPLIFT FORCES

The uplift on the base of dam and in the body of dam caused by water percolating through the pores in rock or dam concrete has been allowed differently by different authorities. The two factors required to be known are the uplift intensity and the area over which it acts. The exact determination of uplift intensity could be carried out only on assumption of a homogeneous medium and applying Darcy's Law of flow. Such a theoretical value would hardly represent the actual uplift pressure distribution, due to

substantial departure from the ideal conditions. The foundation rock is seldom homogeneous, while measures to reduce uplift pressures by providing cut off grout curtains and seepage relief drains are provided.

Some of the dams of the Tennessee Valley Authority have been designed on the assumption of uplift equal to reservoir head at the heel reducing to the tail water head at the drain and this pressure is then taken to be constant to the toe of dam. A few dams of the TVA have also been designed on the existence of average of tail and head water pressure at the drain, the pressure at the heel and toe being the head water and tail water respectively. The assumption of reduction of pressure at the drains, which are provided to reduce the seepage head, is quite logical. The U.S.B.R. has given a relation for determination of pressure at the drains, in terms of other known quantities. Actual measurement of uplift on existing dams has shown very low uplift pressure in the neighbourhood of drains. Theoretical computations based on a homogeneous medium indicates no pressure at the drains. However, most of the dams of U.S.B.R. have been designed on the assumption of no reduction in uplift pressure at the drains, the uplift varying uniformly from the head water at the heel to the tail water at the toe, acting on 2/3rd of the total base area.

Uplift pressure at the base of Rihand Dam was assumed in a number of ways and it was found that the criterion of triangular distribution of uplift pressure acting on 2/3rd of the base area gave the worst effect, which method has accordingly been used in the design. A cut-off curtain inside rock being provided along the foundation grouting and drainage gallery, while seepage relief is being provided by making 3″ drainage holes 40 to 60 feet deep at 10 ft. centres, immediately downstream of grout curtain and draining into the foundation gallery.

The same distribution of uplift pressure has been taken on horizontal planes through the dam concrete which is rather severe since actual measurement on some of the dams in U.S.A. indicates no uplift pressure in the body of the dam. Six inch dia. formed openings are being made at 10 ft. c/c and at a distance of 9′-6″ from the base line extending vertically from the foundation gallery to the top of dam and draining into the foundation gallery, to reduce uplift in the body of the dam. Various forces acting on the dam are shown in Fig. 6.

CRITERION FOR STABILITY

The stability of a gravity dam is generally based on the following:—

(1) The dam should not overturn.
(2) The dam should not slide on any plane, through the base or through the foundation.
(3) The dam should have safe stresses.

The first criterion of stability is generally more than covered by the criterion for safe stresses. In order that a dam may not slide on any plane, the total forces tending to slide the dam should not exceed a certain ratio of the normal forces on the plane. Expressing mathematically $\frac{H}{V} \angle$ 'f'., where 'f' is the coefficient of friction. The safe value of 'f' adopted for Rihand is 0.65 and under abnormal loads it is allowed to exceed upto 0.75. These assumptions are in accordance with the standard practice. The shear friction factor $(fV + r^{SA})/H$ allowing a shear strength 'S' of 600 psi and value of 'r' equal to 0.5 has also been calculated, the minimum value for the factor being taken as 5. In emergency, this has been allowed to go down to 4.

The criterion of safe stress in the dam was first of all enunciated by Levy, who specified that the stress at the upstream face of dam at any point should not be less than the water pressure at that level. This remained the criterion for stress at the upstream face of dams for long. Design assumptions became bolder as the dam construction advanced and the criterion of absence of tension or even permitting some tension at the upstream face, has been progressively coming in vogue. Very bold assumption for safe stresses has been made for the Grand Dixence Dam where the resultant is allowed to deviate to the ends of middle half during abnormal loading.

The actual stresses worked out for the normal loads for the non-overflow section are shown in figure 7. These stresses have been worked out by gravity analysis based on elastic theory and linear distribution of vertical stress.

The stresses at the toe have been brought within permissible limits, by the provision of fillets. The size of these fillets has been calculated on Brahtz's theory, the height of the fillet above the heel and the toe being given by the relations:—

$$\text{For heal Height of fillet} = \frac{-H^2}{5} + 6H - 3.5$$

$$\text{For toe; Height of fillet} = \frac{-H^2}{3.0} + 6.5\ H - 3.0$$

where H is in 100 ft.

Circular fillets could not be constructed easily and hence sloping fillets with 2 horizontal to 1 vertical at downstream and 1 horizontal to 2 vertical on the upstream have been provided.

The stresses so calculated shall be affected by temperature variation caused by heat of hydration of cement and variation in atmospheric temperatures. The variation in stress due to heat of hydration of cement is minimised considerably by precooling while the effect of external variation of temperature is confined to a very shallow depth of concrete at the face of dam and this effect is not considered in the design computations. Large variation in stresses near the face has been actually found to occur during stress measurements on Shasta Dam. Such high stresses are accounted for by providing higher

strength concrete (4000 psi against 2500 psi for interior) in a depth of 5 feet. The block joints are chamfered to avoid cracking at the edges.

SURPLUSSING ARRANGEMENTS

The surplus water would overflow on the dam through thirteen bays of forty feet each, controlled by radial gates 28 feet high. The crest level has been fixed at elevation 852.00 from economical considerations and the overall length of overflow section including the piers is 624 feet. The piers are eight feet wide except two 12 feet wide piers vertically above the river outlets. The overflow profile of the spillway comprises of two portions, one lying upstream of the crest and rising to it, while the other is the falling portion downstream of the crest. The shape of the spillway profile has been adopted differently by different authorities. The three most common profiles are shown in Fig. 8. Extensive model tests have been carried out for the profile and all the three give desirable flow conditions. The Rihand profile has been provided with an over-hang, designed in accordance with the U.S.B.R. specifications. The profile with overhang gives a greater area of approach to the crest and has a slightly higher coefficient of discharge.

In respect of the geometry of spillway profile, an attempt is made to fit in the profile to the underside of an overflowing jet from a sharp crest. Since this is possible for one particular head over the crest only, a number of practices are forthcoming. The Army Corps of Engineers, U.S.A. have recently adopted the practice of designing the profile for 75% of the maximum possible head on the crest. This results in sub-atmospheric pressures for the design head causing an increase in the discharge. Sub-atmospheric pressures have not been considered desirable in the past, since these cause:—

(i) an additional overturning mement on the dam.

(ii) vibrations in the structure which may be harmful and,

(iii) an absence of similarity between the model and the prototype.

The U.S.B.R. practice is to design for the full design head over the crest as well as for a jet issuing under the gate when it is raised by one foot.

The Rihand profile have been designed for the design head of 34 feet. To minimise negative pressures during part gate openings when the jet has a tendency to leave the surface, the sill of the radial gates has been shifted downstream of crest by 7′-4 7/32″ horizontally giving a 1 foot lower elevation of sill. This shifting of the gate would give a downward trend to the jet issuing from under the gate. The shift is also necessary to avoid interference in the rise of the radial gate with the spillway bridge. The pins for the radial gates are located on the piers at elevation 860.00 and the piers have been designed for the unbalanced thrust transmitted by the inclined arms of the radial gates when one bay is overflowing, while the other gate is closed.

The stability of the crest has been tested for the construction condition, for gates in the lowered position and water upto the top of gates, and for water flowing over the crest. For the construction condition steel has been provided in each lift to take the load of the green concrete in the next lift and a live load of 100 lbs. per sq. ft. For the case when water is held by the gate, the crest section is safe and piers have also been tested to be safe for the total horizontal thrust combined with earthquake. The drag due to highly velocity flow on the crest surface is very low. Richer concrete is provided in the region of the crest in one foot depth, and finish will be done by vacuum pads.

BUCKETS

Water flowing down the spillway would acquire a velocity of 125 feet per second which will impart very high energy to the flow. The dissipation of this energy shall be obtained by providing a bucket at the toe of the overflow section. The sound rock levels at site warranted a stepped construction of the bucket rising in stages towards the left bank and such a construction would have minimised excavation in sound rock and would have been most economical. The model tests on stepped bucket construction, however, indicated flow across the step and other complications due to non-uniformity of flow. Satisfactory performance could be obtained by providing divide walls at the step, which added to the cost of construction considerably. The most economical elevation to give a minimum total cost of rock excavation and concrete on the bucket invert, if provided throughout at one level, was found to be 625.00 and model test for such a bucket showed very satisfactory performance. The bucket was therefore adopted.

A 3 feet wide closure block has been provided between the overflow dam and the bucket concrete. Concrete in the closure block shall be poured two to three months after placing the adjacent concrete.

The high velocity jet while traversing the bucket causes a drag on the bucket surface. The drag force 'F' in lbs per sq. ft. is given by the relation $F = C \cdot \frac{WV^2}{2g}$ where W is the unit weight of water, V the velocity and C = 0.0024,. The value of F works out to only 37.5 lbs. per sq. ft. which does not warrant any provision of reinforcement. The horizontal force on the bucket as a whole due to the change in momentum of water from the invert to the lip of the bucket is resisted by the entire depth of the bucket concrete and has insignificant effect.

Surface reinforcement has been provided by 1″ diameter bars at right angles to the base line and ¾″ diameter bars parallel to the base line both at 1 ft. centre to centre. The bucket concrete has been an-

chored to the foundation rock by $1\frac{1}{4}''$ dia. bars at 5'c/c to safeguard against uplift, corresponding to maximum reservoir and tail water levels, 50% of the dowels being continued to the surface mesh for obtaining support and to bind together various concrete layers. The effect of dynamic forces of water in the bucket as a stabilizing force against uplift has been neglected. A system of drains has been provided under the bucket concrete to reduce the uplift. These drains open into the side walls and downstream of the bucket lip. Reduction of uplift forces on the bucket concrete due to relief by the drains has not been allowed while designing the anchors, to be on the safe side. The bucket lip edges are protected from erosion by putting in steel plates.

SPILLWAY TRAINING WALLS

The training walls confine the flow on to the downstream face of the spillway and are six feet wide with a free board of five feet above the overflow profile for the design head. Exact determination of overflow profile on steep spillway slopes could not be done mathematically. Use could be made of Manning's formulae which is the only available relation for velocity determination in terms of slope. Determination of profiles by model studies does not give an exact picture of the prototype flow due to air entrainment, which occurs at a velocity of 20 feet per second and above. It has been established by experiments that the air entrainment is proportional to Froude's number and is roughly given by $m = \frac{V^2}{200\ gD}$. Where m is the percentage of air entrained, V is the velocity of flow in feet per second and D is the depth of flow. The over-flow profile could thus be worked out by Manning's relation, using a rugosity coefficient equal to 0.008, and increasing the depth of flow on the basis of entrainment of air worked out by the above relation. The U.S.B.R. instead recommends using a value of rugosity coefficient of 0.014 to obtain the air entrained profile. The profile of overflow for Rihand Dam has been found out accordingly. The height of wall has been reduced towards lower levels, being 18 ft. at R.L. 808.23 and $14'\text{-}10\frac{1}{2}''$ at 683.00, the slope of top of wall being 0.67 horizontal to 1.0 vertical.

The structural design of the wall is based on considering it as a cantilever from the dam and subjected to water pressure on one side (Fig. 9). The ements are calculated in a plane normal to the downstream face of dam, the component of water load normal to the dam face being taken as the effective overturning force. Seismic forces in the transverse direction have also been considered. Reinforcement has been provided in the wall normal to the face of dam, normal working stress being taken for seismic and water load acting separately and emergency stresses for the two forces acting together. Expansion joints have been provided at forty feet intervals along the face of dam. To obviate the creation of undesirable effects due to differential thermal effects in a six feet wall and mass concrete in dam, the wall shall not be poured monolithic with the dam. Instead keys shall be provided in the mass concrete and the whole reinforcement shall be dowelled.

SPILLWAY RETAINING WALLS

The retaining walls have been provided as extensions of the training wall. The alignment of these walls was tested extensively at the Irrigation Research Institute, Roorkee, to obtain scour conditions downstream of bucket and deposit of debris in the tail race for different grades of movable bed as well as for unerodable bed corresponding to the sound rock at site. Very long retaining walls were indicated on model tests in the initial stages to save the right bank of the river from erosion and to attain proper flow condition in the tail race. The length downstream of the bucket was reduced subsequently to 125 feet on the east side and 50 feet on the west and satisfactory flow conditions were obtained, the smooth conditions in the tail race being assisted by the introduction of a regulator. This regulator has been provided for an additional facility for isolating the draft tube. However, to provide for a future necessity of extension the retaining wall between the tail race and the spillway, a further 100 feet length of the retaining wall shall be made with its top above tail water level. The retaining walls have the top at elevation 638.00, i.e. 3 feet higher than the maximum tail water level. (Fig. 10).

Structurally, the walls are designed as vertical cantilevers rising from the foundation rock. The portion of the wall in the bucket confined the curved flow of water and is thus subjected to a dynamic force in addition to hydrostatic force. The existence of dynamic force on the walls is supported by observation on piezometers fixed to the models. The retaining wall downstream of the bucket shall remain lower than the profile of water during maximum overflow conditions. It has however been found on models that this high water has no tendency to spread sideways and the wall is not overtopped. Piezometric pressures in this region indicate very low pressures as compared to the level of water, due to convex flow and high air entrainment. Water on the tail race side being upto the level of the tail water, the retaining wall gets more load from the tail race side inspite of the higher water level on the spillway side. To confirm the existence of low pressures in the prototype, piezometric tubes have been installed in the retaining wall. The design of retaining wall has been based on the weight of water on the spillway side being taken as 62.5 lbs. per cu.ft. upto a height of 25 feet and 50 lbs. per cu.ft. above it. Uplift has been allowed on the base of retaining wall and seismic forces in the horizontal direction are taken equal to one tenth of gravity. Dowelling of the walls into the foundations is

required for safety and these dowels allow 1/8″ extra for corrosion. Richer concrete has been placed on the faces of the retaining wall.

OPENINGS WITHIN THE DAM

The openings in the dam could be classified into two types: the hydraulic openings and other openings. The hydraulic openings are connected to the upstream reservoir and through these flows water, either at low velocity and high pressure as in the case of penstocks or at a high veocity and little pressure as in the case of outlet conduits, or else water is simply impounded in the opening such as in gauge wells or the penstock and sluice airvents. Other openings comprise a number of galleries inside the dam and vertical shafts inter-connecting the galleries and the galleries with the top of dam.

The design of hydraulic openings is governed by (i) suitable hydraulic conditions and (ii) structural stability. The suitability of hydraulic conditions would depend upon the requirements of flow, while structural stability is attained by analysing the stresses around the opening and providing reinforcement where required.

PENSTOCKS

The penstocks provide a passage for water from the reservoir to the turbines. Minimum loss in head should occur during flow of water through the passage. Vortices should not be caused to occur in the passage since such vortices travel upto the turbine and reduce the power available. The most economical diameter of the Rihand penstocks has been found to be 16 feet and the alignment has been guided by following considerations, which are the normal standards adopted by the U.S.B.R.

(1) A length of 5 to 10 times the diameter of the penstock should be horizontal from the centre line of the unit, so as to have a uniform distribution of water in the scroll case.

(2) The radius of curvature in the penstock profile should be three to five times the diameter of penstock.

(3) The entry into the penstock at the upstream face of dam should be sufficiently lower than the dead storage level. The minimum height of water above the penstock inlet is generally specified at 0.3 times the height of opening at inlet.

The best turbine setting is obtained with centre line of runner at elevation 635.05 which becomes the elevation of the penstock at its joint with the scroll case. The penstock is then horizontal upto 80 feet from the centre line of turbine, where a curve of 6 feet radius turns it to 35 degree to horizontal, and this slope continues to the inlet. The centre line of penstock intersects the upstream face of dam at elevation 734.69. A seal of 29.63 feet is obtained upto the minimum reservoir level of 775.00, which is ample for the opening of 27.15 ft. height.

The penstock being at an angle to the normal to the upstream face, the entry into the penstock is unsymmetrical and suitable inlet transition to induce the turn in flow and proper flow conditions has to be provided. The inlet transition profile depends upon the arrangement for intake emergency gates. Some of the Indian dams designed by C.W.P.C. have been provided with intake emergency gates inside the dam, two transitions being provided one from the inlet section to the gate section and the other from the gate to the circular penstock.

The usual practice in U.S.A. is to provide the intake emergency gate on the upstream face of dam when only one transition from the rectangular inlet opening to the penstock is provided. Inlet transition for Rihand penstocks are also based on this practice and have been designed to correspond to the shape of a free jet from an orifice to its *vena contracta*, the orifice being at the upstream face and *vena contracta* at the penstock. The curves for reduction in area have been experimentally found out by various experimentors. An additional reduction in the area is required to induce sufficient acceleration in flow for the turn. Assuming a coefficient of contraction, with the side contractions suppressed, of 0.60, and the inclination of centre line of penstock being θ to the horizontal, the area at inlet is fixed as "penstock area". $\frac{}{0.6 \text{ Cos } \theta}$ The profile of the transition from the rectangular inlet to the circular section of the penstock has been made to conform to the boundary of the free jet from the inlet to the *vena contracta*. The contraction curve is given by the ellipse

$$\frac{X^2}{(1.1D)^2} + \frac{Y^2}{(0.291D)^2} = 1$$

The major axis of the ellipse for the top profile is kept horizontal, the top line of the penstock meeting ellipse tangentially. In the case of bottom profile the ellipse has the major axis paralel to the centre line of penstock, the bottom line of penstock being tangential to the ellipse. The profile is made tangential to the upstream face of the dam by providing a circular curve of radius $\frac{b^2}{a}$ between the elliptical profile and the upstream face of dam. Adopting the elliptical curves for the vertical profiles, the height at the inlet is fixed and consequently the width. The variation from the width at inlet to the penstock diameter is linear. The sides of the transition are extended upstream of the dam elliptically, thus suppressing the side contractions and providing space for the slots of intake gate and stop logs. The variation of the area of transition from the inlet to the penstock section is in proportion to the reduction of area from the orifice to the *vena contracta*. The centre

line of the flow has been taken as the locus of the centres of chords of circles touching the top and bottom profile of the transition. The centre line of transition has been divided into equal parts and the height, and width of sections represented by chords passing through these points being fixed already at each section, in accordance with the area reduction curve for a free jet, the requisite area of section is provided by elliptical fillets at the four corners. These fillets gradually become bigger and bigger till the circular penstock section is obtained.

AIR VENTS FOR PENSTOCK

An airvent has been provided just downstream of the penstock emergency gate to reduce the vacuum head on the gate when the gate is suddenly closed, and to exhaust air when the penstock is filled in for balancing the head on the two sides of the intake gate. The quantity of air required to be admitted during sudden closures has been fixed equal to the free discharge of water in the penstock with the intake gate closed and the turbine wicket gates fully open. This quantity of air has to be supplied without developing high vacuum. The permissible limit of vacuum has been fixed at half the atmosphere. The area of air vent which would admit the required quantity of air without exceeding the negative pressure of half the atmosphere is worked out by different available methods and fixed at a higher figure of 36 inches. This size of airvent gives a negative head of 6.15 ft. at the gate.

The penstock end of the vent opens at R.L. 742.50, while the top of vent is provided with centre line at R.L. 887.50 which would not permit outflow of water through the air vent even in the event of reservoir being at elevation 886.00. The air vent opens on the downstream face of dam.

OUTLET SLUICES

The criterion of good hydraulic conditions in the case of sluices is the absence of negative pressures, and sudden variation in flow which may lead to cavitation. The tools for the study of water flowing under high velocities being still inadequate, the design of proper alignment for the outlet sluices is generally carried out by model tests. The Rihand sluices were no exception and extensive model tests were carried out at the Irrigation Research Institute, Roorkee, to determine the following:—

(a) Whether the sluices had sufficient capacity.
(b) Whether pressures in the full length of the conduit were positive for various gate openings thus assuring the absence of cavitation.
(c) Whether the discharge from the sluice had satisfactory dissipation of energy both when spillway and sluice are running together and when sluice runs alone.

Model studies were based on providing two outlet sluices, each of capacity 2,500 cusecs, so as to obtain small standard size conduits which would require smaller control gates and which will cause lesser concentration of stresses. Sluice profiles with inlet at 770.00 and 710.00 were tested on models. (Fig. 11).

The absence of cavitation in outlet sluices can be assured, if the hydraulic gradient line lies wholly above the conduit roof. Irregularities in the flow also tend to cause separation and if the sluice surface presents a minimum of such irregularities, cavitation will not occur. Gate slots are a necessary evil as they provide the irregularities in the boundary of flow. Another type of irregularity in the flow, which is most conducive to negative pressures, is the departure of the boundary of flow from the natural flow pattern. In such circumstances, the flow is forced to correspond to the boundaries as against the natural tendency to flow otherwise.

The alignment for the sluice with inlet at 770.00 was kept horizontal from inlet to a sufficient length, a circular curve being provided at the end to have the outflow tangential along the downstream face. The circular curve in the profile forced the sluice discharge to take a turn and a tendency for separation at the begining of the curve existed, causing negative pressures. Such negative pressures were eliminated by providing a step in the floor and providing an air vent. The section of the sluice worked out to $11' \times 5'$ and required a bigger size gate, though the design head on the gate was less. The arrangement was considered unsatisfactory.

A number of alignments were considered for the sluice with inlet at elevation 710.00. The sluice at this elevation is required to be $4' \times 9'$ in size. The various layouts tested on models are:—

(i) Sluices having constant cross section, the invert being horizontal upto 91.75 feet and then curved to meet the downstream face of spillway tangentially.
(ii) Sluice having constant cross section, the invert sloping uniformly from the inlet to the exit end at a slope of 31.56 horizontal to 12 vertical.
(iii) Sluice having constant cross section upto the control gate and an enlarged section $5.5' \times 11'$ downstream, the invert aligned as a trajectory for the jet issuing from the inlet orifice, but rotated as a whole through 5°-27'-36" to meet the bucket tangentially.
(iv) Sluice having a gradually reducing section of the conduit from the inlet to the exit end, the invert being provided along the trajectory.

Model tests indicated most satisfactory performance for the sluice having invert along a trajectory and section being reduced gradually. Enlargement of the section downstream of control gate caused negative pressures upstream of the gate at full gate opening. A reduction of the sluice section at the gate re-

moved the negative pressures but the discharge was reduced. The sluice with gradually reducing section had a reduction of 13.7% the rate of reduction being as occurring in a free jet due to increasing velocity along the flow under gravity and neglecting friction. All through positive pressures were obtained with this alignment, which assures no tendency for cavitation, specially at the gate guides. Sluices with gradually decreasing section have been adopted for Rihand dam.

Elliptical transitions have been provided at the inlet in accordance with the recommendations of Army Corps. of Engineers, U.S.A. The inlet dimensions are 6′-8″ by 14′-2¾″ giving an area 2.64 times the sluice area. The inlet has been found satisfactory on models, the discharge co-efficient being 0.836. The control for the sluices has been provided by a slide gate inside the body of the dam and a fixed wheel emergency gate at the upstream face. The emergency gate shall be lowered through a slot in the spillway pier which has been made wider to allow for such a construction.

AIR VENTS FOR SLUICES

Air vents are provided to give a passage for the movement of air to and from the outlet conduit. Such passage is required for:

(i) Replacing the air sucked out by the hydraulic jump, if formed in case of partial gate openings.

(ii) Supplying air as the hydraulic jump moves down the sluice when the gate closes,

(iii) Exhausting air as the hydraulic jump moves up when the gate is opened.

(iv) Providing air in a pocket between two jets.

The air vents for the first three functions are located downstream of the emergency gate at face of dam and the control gate inside the dam whereas the air-vent for the fourth function is placed at the exit end to provide air in the space between spillway and sluice jets.

Failure of the air vent to function properly may give rise to negative pressures inside the conduit, which might cause vibration of gates and cavitation. The capacity of airvent, therefore, has to be such that it does not fall short of requirements in the most critical conditions.

Sub-atmospheric pressure is created downstream of the control gate for partial gate openings when the gates are operated under high heads. Largescale tests carried out for 4′ × 6′ inclined conduits at Norfolk dam for operation under a head of nearly 160 feet, indicated that sub-atmospheric pressures are created when gate opening is less than 90% of the gate height. Critical conditions were obtained for the gate openings of 15% and 80%. Negative pressures were found to occur at 6 ft. to 7 ft. downstream of the regulating gate and the maximum pressure reduction with a 20″ circular vent downstream of gate, was measured as 8 to 9 feet of water. The capacity of 20″ diameter airvent, having 9.1% of the gate area, works out to 525 c.ft. per second at a suction head of 8 feet, which equals to 24% of the discharge through the sluice at full gate opening.

A number of arbitary relations are available for fixing the area or the capacity of the air vents, while formulae have also been developed by a few investigators.

Based on these available criterion, and somewhat on the safer side, the Rihand dam sluices have 12″ air vent downstream of emergency gate, 30″ air vent downstream of control gate and 12″ air vent at the exit end. The air vents are open to atmosphere at top through the spill-way piers.

DIVERSION CONDUIT

The diversion conduits, being required to run during the construction period only under low heads, the flow conditions were not a major guiding factor. A 12 feet wide and 14 feet high horse shoe shaped conduit without inlet transition has been provided centrally in block 37. This conduit will be plugged as soon as the permanent river outlets are available for diversion and arrangement for proper back filling has been made. Shear keys have been provided and grout pipes have been laid for future grouting of the joint between the plug and the dam concrtee.

GAUGE WELLS

Still water surfaces at the same elevation as the reservoir and the tail water, which would not be affected by waves and wind are obtained by providing, 2′6″ dia. vertical shafts inside the dam in block 34, connected to the reservoir and tail water through pipes. Automatic gauge recorders with floats shall transmit the gauge readings on to the Power Station Control Board. Valves are provided for draining out the wells and cleaning the pipe line.

SYSTEM OF GALLERIES, SHAFTS AND CHAMBERS

Galleries have been provided to enable grouting and drainage of dam foundations and providing approach to the chambers inside the dam where equipment is placed for controlling the flow in various hydraulic openings. Vertical shafts interconnect various galleries and provide approach from the outside of the dam and these will house dog-legged staircase and elevators.

The clear concrete width from the upstream face of dam to the openings has been kept at a minimum of 10% of the head of water at that level from consideration of seepage into the opening. The minimum distance of openings from the contraction joints has been fixed at 5 feet.

STRUCTURAL DESIGN OF OPENINGS

Openings inside mass concrete cause a redistribution of stress in the body of the dam, which results in heavy concentration of stresses around the opening. The exact analysis of such concentration is not possible, as an opening inside the dam presents a three dimensional problem. The problem could be tackled by taking sections through the opening in various directions, which results in a two dimensional problem of an opening in a plane. The handicap still remains since the stress distribution for openings in a plane could be obtained theoretically only for a circular or elliptical opening in an infinite plane having uniform stress. The plane in the case of a dam could hardly be considered infinite and the stress is never uniform. Openings are also in some cases located close to the boundary of the plane.

Most of the openings in the dam are rectangular in section. Stress patterns for rectangular openings have been determined by photoelastic studies, and coefficients have been worked out for the total tension and compression existing at the boundary of the opening for unit primary stress in the plane. Such studies have been made for different ratios of the two sides of the opening. Circular openings close to the boundry of the plane have also been analysed by photoelastic method. It has been found that the stress pattern around the opening near the boundary of the plane becomes the same as that of an opening existing in an infinite plane, when the opening centre is located at a distance of three times the diameter of the opening. Circular openings close to the boundary could also be studied mathematically by considering a circular plate having an eccentric hole, while the radius of the plate becomes infinite.

To obtain the pattern of stress concentration around an opening, the section plane of the opening is assumed to have a constant uniform stress corresponding to that at the centre of the opening, calculated on the assumption that the opening does not exist. The value of the stress at the centre of the opening is worked out by gravity analysis of the dam section.

The stress concentration around the openings is calculated in accordance with the following procedure for different openings.

(1) The stresses around the gallery opening have been worked out by the use of U.S.B.R., Engineering Monograph No. 12, which gives the stress pattern for a gallery of similar shape.

(2) The stresses around other rectangular openings away from the boundary of the dam have been worked out by the use of coefficients given in U.S.B.R. Technical Memorandum by H. Boyd Philips. Stress concentrations around rectangular openings have also been worked out by considering an equivalent circular opening having its centre coincident with the rectangular opening, and increasing the total stress by 30%, to account for the effect of sharp corners in rectangular openings. In a few cases analysis has also been done on the basis of an inscribed elipse and increasing the total stress by 30%.

(3) The stresses around circular openings have been worked out in accordance with U.S.B.R. Technical Memorandum No. 457.

(4) For openings close to the boundary, stresses around circular openings have been worked out by the use of U.S.B.R. Tech. Memo. No. 597. Where openings are rectangular in shape, the analysis has been based on an inscribed circle placed tangential to the opening near the boundary, the stress concentration being increased by 30%.

The design of hydraulic openings has to account for the internal pressures in the opening due to head of water, shrinkage in concrete and a temperature gradient caused around the opening due to cooler water passing through the conduit as compared to the temperature of mass concrete in dam. The effect of these factors is to create tensile stresses all around the opening which have been allowed in the design computations in accordance with U.S.B.R. Tech. Memo. No. 457.

Reinforcement around the openings in Rihand Dam has been provided for the total tension around the opening, assuming that the concrete takes no tension whatsoever. The compressive stresses around the openings are generally not very high as to require reinforcement in concrete; higher strength concrete being specified around the openings in a distance of three feet. Where higher compressive stresses are met, reinforcement is designed accordingly. A nominal reinforcement not less than $\frac{1}{2}''$ dia. bars at 1′ c/c however, has been provided in the compression region of the openings as well. Steel in tension zones is provided at a minimum of $\frac{3}{4}''$ diameter bar at 1′ c/c.

A question has often arisen whether the steel liner in the hydraulic openings should be considered as a reinforcement or not. The question is most pertinent in the case of penstocks which are practically lined all through. (The outlet conduits of Rihand Dam are lined only from the inlet to the control gate, while the penstocks are lined from downstream of transition to the exit end). Varying design assumptions have been made from time to time. Fontana Dam, constructed during the War period, when structural steel was in short supply, used the criterion that the steel liner will have a stress of 8,000 psi and the reinforcement, 16,000 psi for the upper 198 feet of the conduit. Shrinkage forces were assumed at 120,000 pounds per linear foot of conduit. The shrinkage effect was allowed totally in the reinforcement bars and the stress allowed to go upto 30,000 psi. Tension of 75 psi was allowed in the concrete as well. For the lower 184 feet of the conduit, the full water load was carried in the plate, while reinforcement was provided only for stresses caused by volume changes.

Strain meters were fixed to the steel liner and

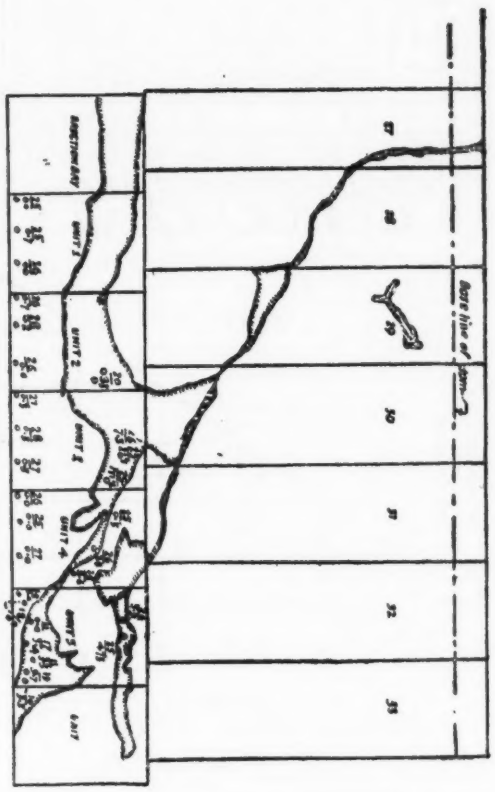


FIG. 2

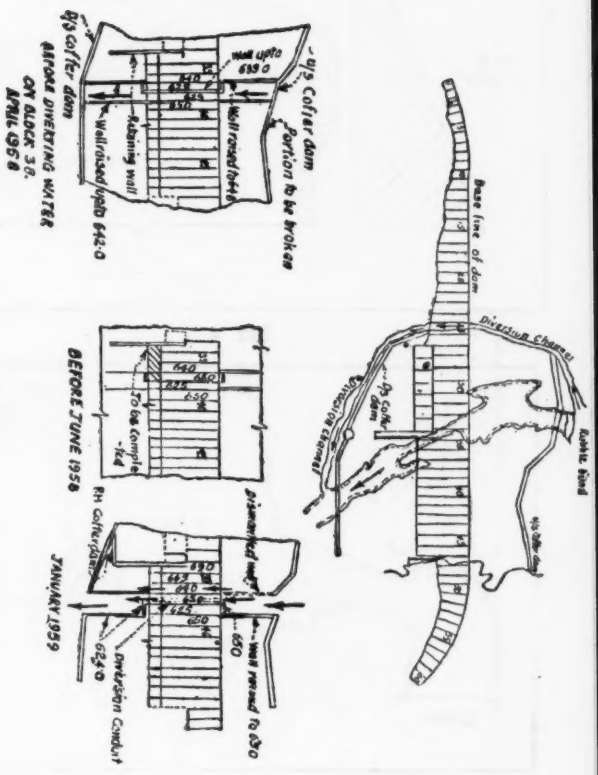


FIG. 1

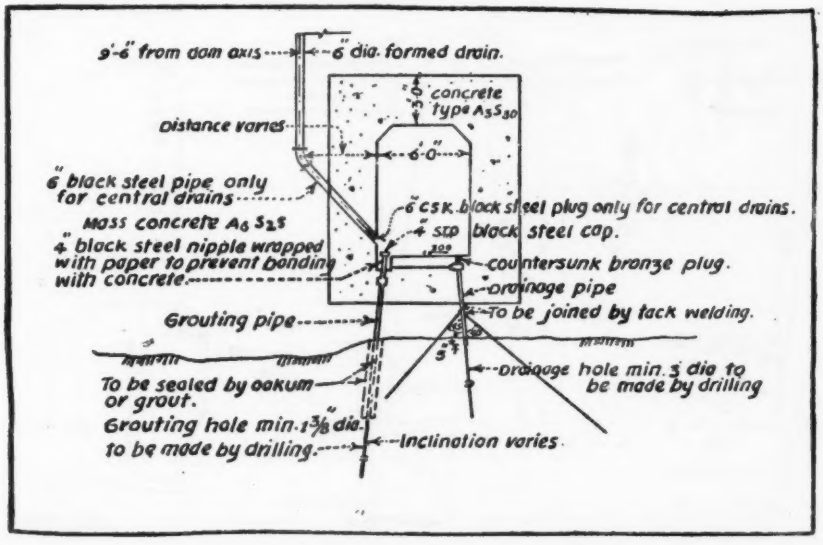

9'-6" from dam axis
6" dia. formed drain.
Distance varies
6" black steel pipe only for central drains
Mass concrete A6 S2S
4" black steel nipple wrapped with paper to prevent bonding with concrete
Grouting pipe
To be sealed by oakum or grout.
Grouting hole min. 1 3/8" dia. to be made by drilling.
6" C.S.K. black steel plug only for central drains.
4" STD black steel cap.
countersunk bronze plug.
drainage pipe
To be joined by tack welding.
drainage hole min. 3" dia. to be made by drilling
Inclination varies

FIG. 3

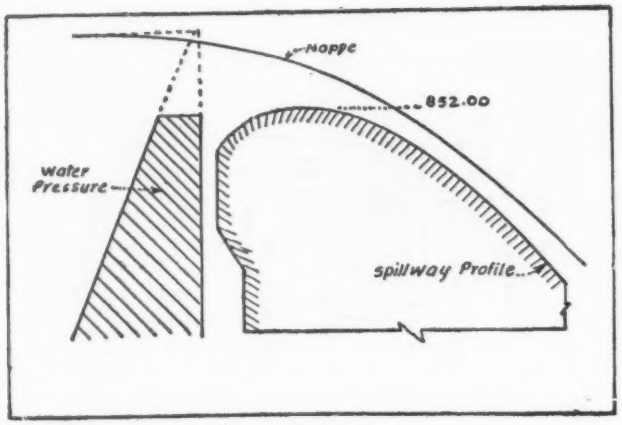


FIG. 4

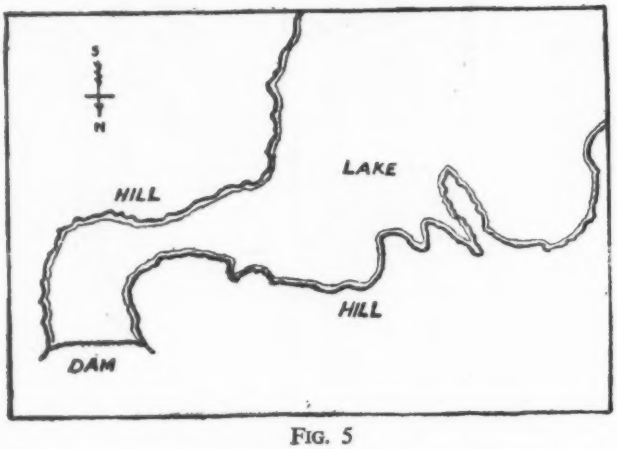


FIG. 5

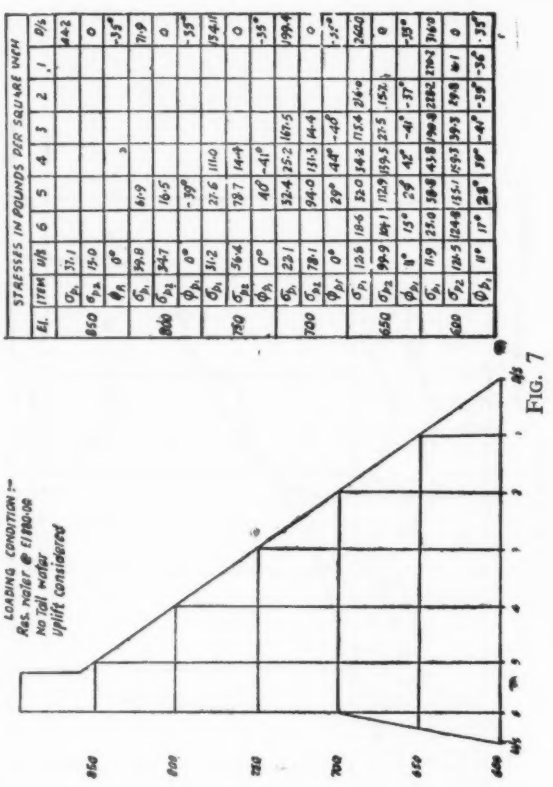


FIG. 7

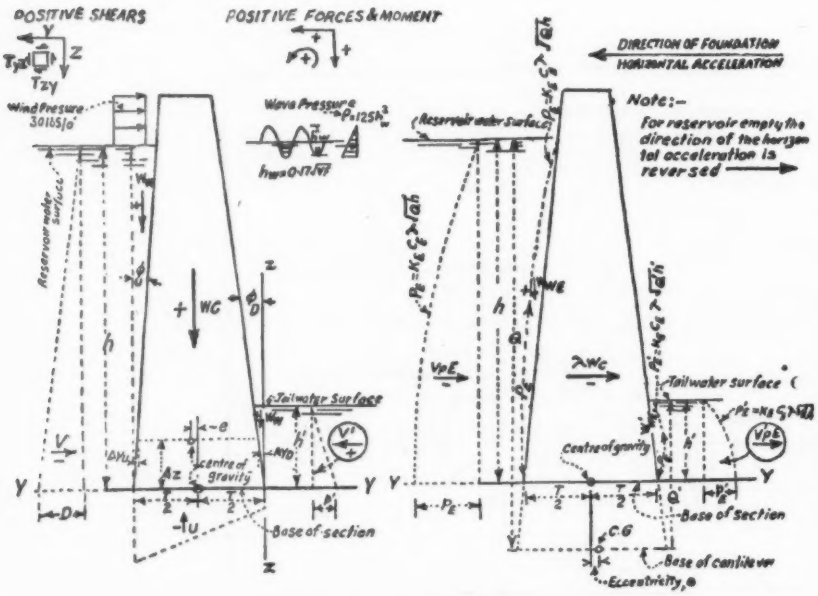


FIG. 6

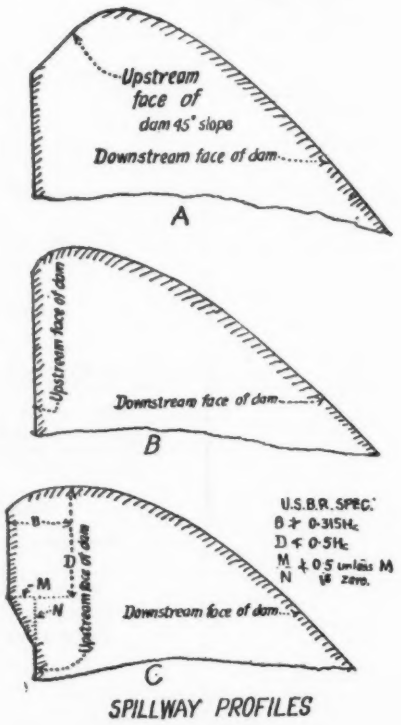
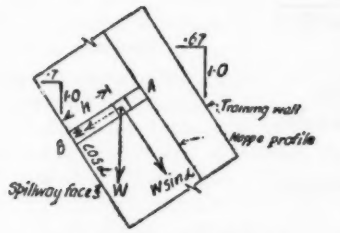


FIG. 8



FORCES ON TRAINING WALL

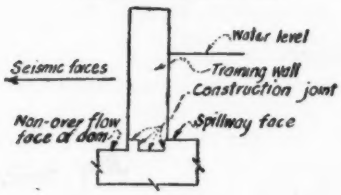


FIG. 9

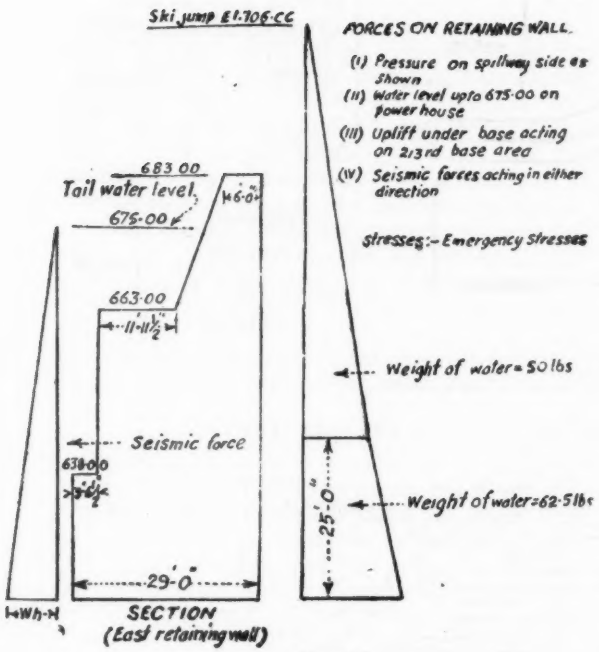


FIG. 10

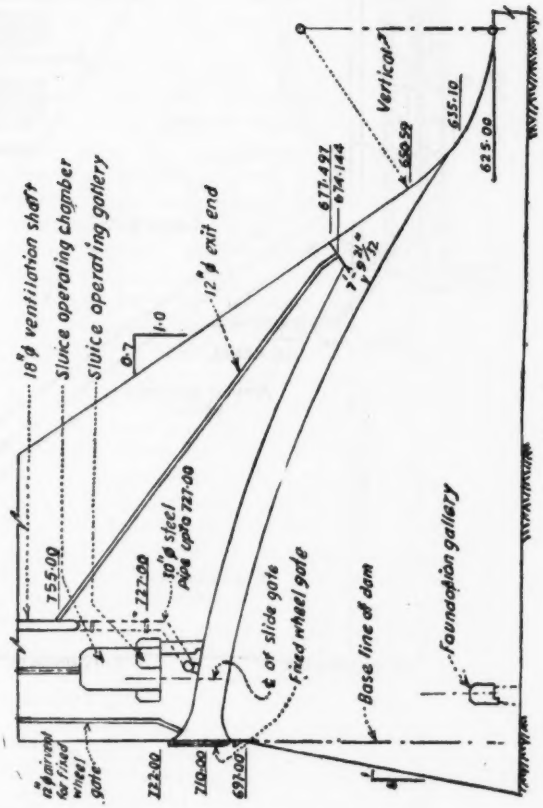


FIG. 11

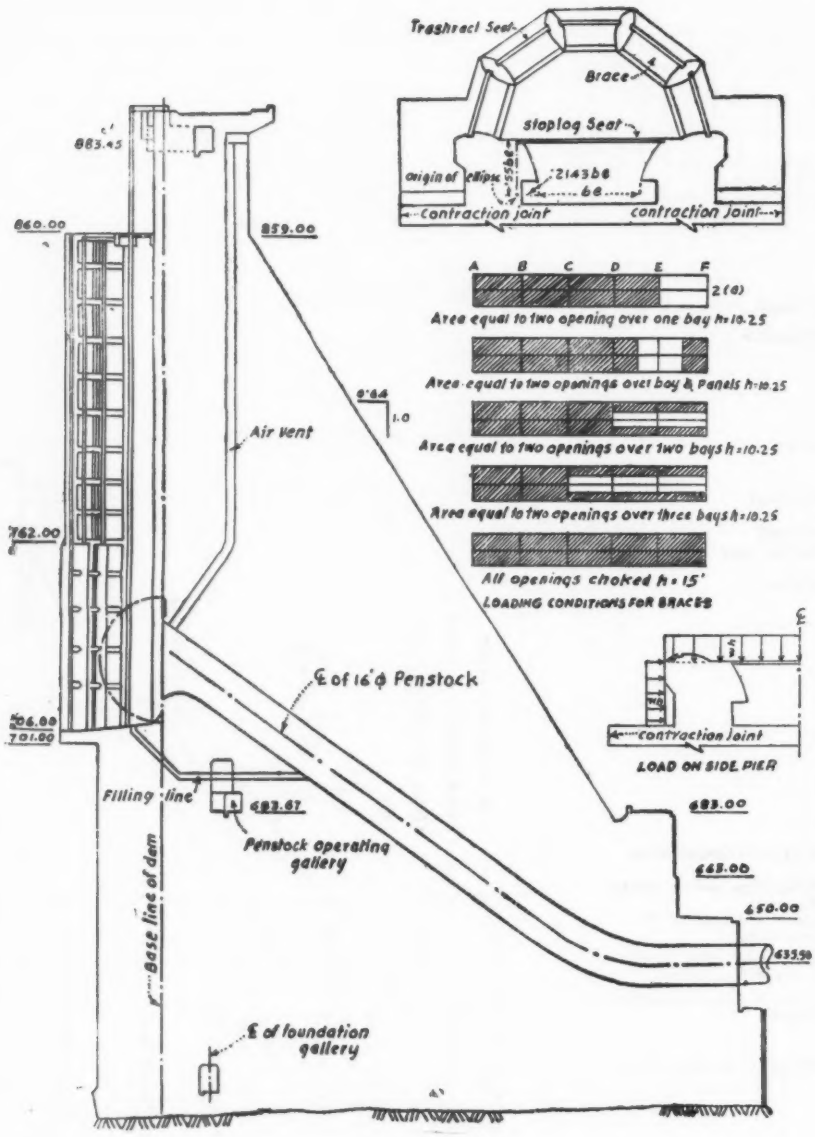


FIG. 12

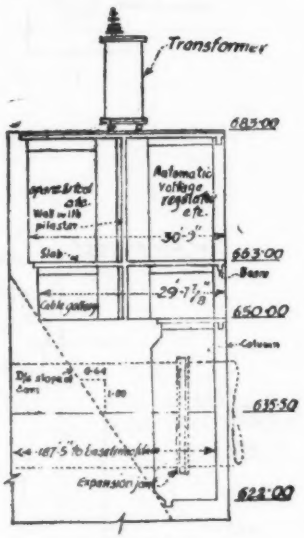


FIG. 13

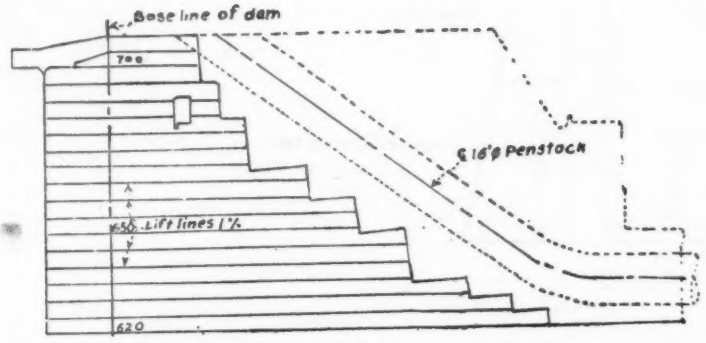


FIG. 14

surrounding reinforcement on the Hungry Horse Dam, though final results of measurements by such meters are not yet available.

The reinforcement around penstock openings at Bhakra Dam has been reduced by 50% of the liner plate area. The same criterion has been used on the Chambal Dam penstocks.

Considering the structural action in general, the liner is embedded in concrete, and any gap between the liner and the mass concrete, which may occur due to shrinkage, is grouted. Moreover, the liner has stiffeners and the reinforcement around the liner is welded at a number of points to the liner, making the liner as a whole rigidly fixed with the concrete. On the applicantion of load from inside or outside, the liner is not free to take the deformations and come fully into play. A joint action of concrete and the liner would occur, resulting in very low stresses in the liner and reinforcement in concrete, till the concrete cracks. If the analysis be made by considering a hollow cylinder of composite materials, the stress in liner would be very low, and the entire tension steel for the opening will be required to be placed in concrete on the assumption that concrete does not take any tension. On the other hand it can be argued that all the liner is at the surface of the opening where the stress concentration is maximum and it should be perfectly correct to assume the entire liner as a reinforcement, if the opening be considered to exist in an infinite plane. Final design practice for the provision of steel is yet to be evolved. Reinforcement required by the analysis of the penstock opening for Rihand Dam has been only nominally reduced, the reduction being equal to 10% of the liner area. The liner has been designed for the hydrostatic head plus water hammer, while the reinforcement around the opening is designed for the hydrostatic head only.

It has often been suggested that considerable redistribution of stress occurs due to plastic flow and the stress concentrations around the openings are actually not so high as determined by photoelastic tests or by theoretical calculations. The design of Bull Shoals Dam openings allows for a 30% reduction in stress concentrations due to such relief. No such reduction has, however, been carried out for Rihand Dam.

INTAKE STRUCTURE

Each penstock has been provided with a cage type intake structure extending upto elevation 860.00, for installing trashracks, stoplogs and intake gates. This level of the reservoir is expected to be attained each year and it would be possible to have annual approach to the trashracks for maintenance. The guides for the stoplogs and the penstock emergency gate will extend to the top of dam and lowering of the stoplogs could be done at any time by a cantilever arm gantry running on the top of the dam. The structure comprises vertical piers and side walls extending out of the dam and interconnected by horizontal braces.

The cylinders for the hyrdaulic operated emergency gate hoists are suspended by a frame resting on the mass concrete at one end and a reinforced concrete beam connecting the side walls on the other at elevation 890.92, while motors and other auxiliaries are placed in chambers inside the side walls of the structure.

The trashrack has been provided in five bays 9′-5″ wide and 56 feet high from elevation 706.00 to elevation 762.00. The sections of the braces and piers upto the height of the trashracks are streamlined. The side walls are also streamlined in a height equal to that of the trashracks, the surface being an elliptical extention of the sides of the penstock intake transition, the major axis normal to base line of the dam being "0.55 b_e" and the minor axis equal to "0.2134b_e", where 'b_e" is the width of section at the inlet. The intake gate guides are located at the upstream face of dam, while the guides for the stop logs are located at the upstream end of the side walls.

To obtain good flow conditions and no obstruction to the streamlines, the piers and the bottom platform have been placed outside a hemisphere of radius $0.8h_e$ (21.72ft.) where h_e is the height of penstock inlet opening. Above the trashracks, the piers have been cut down in section and a breast wall has been provided upto elevation 860.00. The horizontal braces have been spaced 11′6″ c/c against the trashracks and 10′-11″ above it. All the members of the intake structure are designed for a differential head of 15 feet, which corresponds to 95% choking of the rack area. The rack bars transmit the water load to the vertical piers, which are supported on horizontal brace frames. Moments are thus caused in the piers and braces due to the differential head on the rack and breast wall. The application of load due to differential head on the trashrack has been considered in a variety of ways to obtain the worst effect on the horizontal brace frames. The bottom platform projects 30′-1½″ from the upstream face of dam, though the cantilever portion is 10′-7½″ only, since mass concrete has been added in a width of 19′-6″ upstream of base line for stability requirements. For design of bottom platform cantilevers, all the load of piers etc. is supposed to be carried by the cantilever, no transference being allowed due to horizontal brace frames. The side walls have been designed as horizontal cantilevers under the differential head.

The intake gate slot and stoplog slot through the top slab at 860.00 will be covered by gratings. Gratings, whereever required, are designed for a live load of 500 lbs. per sq. ft., so as to be very stiff against deflection under load.

SPILLWAY BRIDGE

The spillway bridge shall be a T-beam R.C.C. or structural steel structure simply supported at the ends. Link slabs of short span shall be given above

the piers. Provision has been made to carry the gantry crane on the bridge, the beams being provided under the crane rails. Only two beams can be provided due to the placing of hoisting equipment for the radial gate on the piers. The design of the spillway bridge shall be finalised after final decision in respect of award of contract for the radial gates and hoists, when it would be definitely known as to what equipment is being received.

ELEVATOR TOWERS

Two towers have been provided just downstream of the roadway on the top of dam. The towers built in reinforced concrete, shall be at either end of the spillway and shall besides providing approach to the elevator and hoist shafts, house utilities for the visiting public. The tower on the power house side has a 20,000 gallon water tank for fire fighting in the transformer area.

GALLERIES UPSTREAM OF POWER HOUSE

A system of galleries has been made upstream of the powerhouse and rising from the dam for locating electrical equipment and cables, the topmost floor being used for the transformers. (Fig. 13). The construction has been planned by providing reinforced concrete columns, beams and slabs, while the downstream face of dam has been stepped to suit the floor levels. The R.C.C. structure has columns rising from elevation 622.00 to 683.00 and a wall with pilasters from 650.00 to 683.00 while beams are provided connecting the mass concrete of dam to the wall pilasters and columns. Two columns, $8' \times 2.5'$ are provided in each block to support the transformers. The R.C.C. frame structure has to be discontinuous at the block joints where double columns are provided. The structure has been designed for the following loads:

(1) Dead load and live loads as specified for each floor.
(2) Seismic forces of g/10 acceleration. The seismic forces have been taken only on dead load and $\frac{1}{3}$rd of the live load.
(3) Temperature variation of 50°F.

The permissible stresses are taken as 750 psi compression in concrete and 16,000 psi for tension in steel for normal loads. These stresses are increased to 1000 psi and 20,000 psi when seismic force is allowed and further increased to 1200 psi and 24,000 psi when both seismic forces and temperature stresses are allowed.

WATER SUPPLY ARRANGEMENTS

Two eight-inches diameter pipes have been embedded in block 23 to supply water to high head pumps installed within the dam. These pumps will supply water to the filtration plant for the colony supply.

In addition one two-feet diameter pipe is being placed on either bank to supply water for the industries which might develop on the two banks. These pipes shall be initially closed by blind flanges. Valve chambers are being provided inside the dam with approach from the downstream.

BLOCKOUTS FOR THE PENSTOCK

During the progress of work it was found that the plates for fabrication of penstock were in short supply and the penstock could not be embedded in the concrete in the normal manner. It was at the same time necessary to raise the blocks upstream of the power house to sufficiently high level so as to avoid flooding of the powerhouse area in monsoon 1958. The dam could have been raised to higher level before monsoon 1958, only by one of the following methods:—

(1) making a tunnel for the penstocks, allowing about 3 feet all round the liner for placement of steel and embedding the liner.
(2) Constructing the dam in a stepped profile, allowing a clear space of 5 feet under the liner.
(3) Constructing the dam with a deep recess to allow placement of penstock and back filling later.

The tunnel profile would have allowed for construction of the dam to higher levels irrespective of the placement of liner. This method could not be adopted however due to likely delay in construction which would have made it impossible to achieve the obligatory levels of the dam blocks. Stepped construction was preferred for expediency in construction (Fig. 14).

To provide proper bond between the two concretes, reinforcement bars have been provided both on the horizontal and vertical faces and arrangement for grouting of the vertical faces has been made. The dowels on the vertical face have been designed to transmit vertical shear in excess of 100 psi on the joint surface which is assumed to be developed by grouting and bond. The topmost lift of stepped construction has been given vertical steel bars to safeguard against the development of tension by a behaviour of dam, as if resting on a steep base. A horizontal mat of steel bars has also been provided in the first continuous lift at the edge of the topmost step to check any tendency of a crack developing along the vertical joint line in the lift below and extending to higher levels. Grout stops have been provided to confine the grout to the desired area and restricting it from reaching the contraction joint.

PREPARATION OF DRAWINGS

Block drawings are prepared for each block of the dam showing all the features in the block and references are given for detailed drawings of each feature. List of embedments required in each block is also prepared and supplied for facility in arranging the various embedments in advance. Drawings for various features are prepared in sufficient detail. Bar sche-

(Continued on page 22)

Control Equipment for Rihand Dam

By M. C. CHATURVEDI

FOR the control and operation of reservoirs of dams or other water surfaces controlled by other hydraulic structures it is essential that suitable equipment should be installed in the structure which may permit controlled flow of water. The purpose of such equipment is dependent upon the function for which the hydraulic structure is built and its location and design is dependent upon many inter-related hydraulic and structural considerations.

The general layout of equipment for Rihand Dam and their salient characteristics and design criteria are discussed in this article. The general description of the Rihand Dam necessary for reference to this article has already been dealt with in other articles. Reference may be made to the design criteria and the specifications and drawings issued from the Rihand Dam Design Directorate for the details.

The control equipment has been divided into following heads for convenience of study.

(i) Equipment for intakes.

(ii) Equipment for outlets.

(iii) Equipment for spillways.

EQUIPMENT FOR INTAKES

General Layout

The general layout of equipment for intakes of Rihand Dam is shown in Fig. 1. Water required for power generation is led from the upstream face of the intake section of the dam through separate sixteen ft. diameter penstocks, embedded in the dam, individually to each turbine of the power house situated downstream at the toe of the dam. At the upstream of the penstocks, gates are provided to stop the flow of the water to the respective individual turbine units, normally for purpose of inspection and maintenance of penstocks, turbines, valves and other equipment. Their main use, however, is in the emergency when the control of turbine units suddenly becomes inoperative, resulting in runaway of turbines. These gates are never used for regulation of supplies to the turbines.

Upstream of the penstock emergency gates, provision is made for the installation of a set of stoplogs for the purpose of inspection and maintenance of penstock emergency gates without taking them out. Only one set of stoplogs consisting of a number of units required to seal off the intake completely from bottom to top of opening is required.

At the upper most end trashracks are installed to protect turbines, etc., from unusually large debris.

Details of the various equipment are discussed as follows.

Penstocks

Salient details of penstocks as used for Rihand Dam are given in Fig. 2.

The alignment of the penstocks and their dimensions are determined from considerations of economy and suitability of hydraulic characteristics. The design should produce steady flow conditions, freedom from excessive turbulance, and satisfactory rates of acceleration and deceleration of flow in accordance with the response of the governor to load changes on the electrical system.

The diameter of the penstocks has been determined from the consideration of economy and has been checked to see that the acceptable velocities specified as 5 ft. per second for low head and 30 ft. per second for high head installations are available. The important factors and elements of cost that varies with the diameter consist of cost of power lost on account of frictional and hydraulic head losses, and capitalised maintenance cost. Other elements such as variation in cost of intake structure, intake equipment and governor etc. can be neglected in preliminary determination of economical diameter. The selection of economical diameter can be conveniently done by the following formula, derived from consideration that for least cost the differential of cost factors expressed as a function of diameter should be zero[1].

$$D = 6.9 \sqrt{\frac{efb\ (5.76392\ K_s\ Q^{2.9}\ L_f + 74.82876\ K_f\ Q^3)\ S_g e_j}{668\ Hav\ L_p\ (1+i)}}$$

Where D = most economical internal diameter, e = overall efficiency of the plant, f = a loss factor corresponding to the load factor under which a plant operates, b = value of power lost by friction and other hydraulic factors expressed in dollars, K_s = coefficient for head lost due to friction, Q = discharge at rated head, L_f = length of pipe used for friction loss, K_f = sum of all constants for hydraulic losses, e_j = a gross allowable tension in steel penstocks, e_j = efficiency of joint of penstock, H = average head on the penstock, a = cost of steel in dollars per lb, r = ratio of annual fixed operating charges to construction cost, L_p = Length of penstock, i = percertage over weight of steel due to laps etc.

An empirical formula, also consistent with theoretical consideration, which was used is

$$D = 4.44 \frac{P^{.43}}{H^{.65}}$$

Where P=rated h.p. of turbine and H=rated head.

From all these considerations the diameter of penstocks was fixed as 16′.

The loss of head for Rihand intake calculated from the standard formula[1] worked out to 1.41′ which is well within the reasonable limits.

The penstock conduits are steel-lined right from the spiral casing to the point where the intake transition ends. Although the liner is embedded in concrete, it is the usual practice to design it for the full pressure head including the hammer head so as to prevent its brusting in case it separates from the concrete at any location. Welded steel penstocks have been used for Rihand Dam on account of economy and efficiency.

Water hammer in penstock depends on the closing time of the wicket gates of turbines, the hydrostatic head and on the length of the penstock. The maximum pressure rise occurs at the wicket gates from which point it diminishes uniformly along the length of the penstock to zero at the intake. It can be calculated according to the standard practice[1].

The fastest specified governor setting is 3 sec., but to cover cases of maximum pressure rise, when closure starts from some relatively small percentage of full stroke water hammer head for a closing time of 2 seconds, has been computed. It works out to 50′ of water head at the wicket gates and reduces uniformly to zero at the intake end.

The penstock liner is designed for two conditions; first when it is subjected to full hydrostatic head and design stresses are normal, and second—when it is subjected to full hydrostatic head plus water hammer head and emergency design stresses are adopted. The penstocks shall be made of weldable steel and accordingly, in conformity with the standard practice, steel of A.S.T.M. designation A 285-47 or equivalent has been specified to be used. A design stress of one half the yield point, which works out to 15,000 p.s.i. is adopted for normal cases. To take into account the embedment in concrete, a design stress equal to two thirds of the yield point, which works to 20,000 p.s.i., has been used, where minimum cover is equal to or more than one half the static head. For emergency conditions a design stress upto two third of the yield point has been used. A corrosion allowance of 1/16″ has been adopted.

Double welded butt joints have been used for all longitudinal joints, which shall be thoroughly radiographed. The joint efficiency has been taken as 100%. The liner is primarily subjected to hoop stress and the liner thickness is determined from this consideration. The thickness works out to 8/16″ at the upstream end and is increased in increments of 1/16″, maximum thickness being 13/16″. No stress relieving is required upto thickness of 1¼″. Stiffener rings 3/4″ thick and 8″ high placed symmetrically at 12′ apart in each liner of 20′ length shall be provided to retain their shape during erection, and embedment. Percolation stops are provided at the intake end as shown in the drawing.

Liner pieces 20′ long are fabricated in shop so that their maximum weight is within the capacity of the cable way. They shall be hydrostatically tested in shop for a water pressure of $\frac{40{,}000t}{D}$ p.s.i. where t is the thickness of the liner and D the diameter in inches.

In addition to the usual accessories, such as manhole, piezometer connections, drain and filling line connection air inlets and outlets, one expansion joint will be installed at the lower end to permit longitudinal expansion and contraction and to allow transverse movement on account of any possible differential settlement of the dam and the powerhouse. The details of the expansion joint are shown in Fig. 2.

Hemispherical steel bulkheads have been provided for closing the sixth penstocks, and for closing the other penstocks also, during installation of turbines etc.

The total tonnage of steel in penstocks, supports, expansion joints and bulkheads is 1169 tons. Penstock liners are being manufactured by M/S Hindustan Construction Co. at site and bulkheads and expansion joints are being manufactured by M/S Wagner Biro of Austria according to departmental specifications and detailed drawings.

Fixed Wheel Intake Gate

At the upstream end of the penstocks fixed wheel gates are provided as described earlier. Their salient details are shown in Fig. 3.

For routine inspection and maintenance the water pressure acting on penstock gate is balanced by closing the turbine wicket gates and filling the penstock with water through filling line before opening the gate, if necessary. In emergency, however, the gate may have to be closed suddenly from full open position in less than 1½ minutes and in this condition the gate closure may have to be done against full unbalanced water pressure. The gates are, therefore, kept suspended, submerged in water, just above the intake opening ready to be lowered immediately. They are suspended from hydraulic hoists which are located at the upstream face at the top of the dam.

The principal features affecting the design of the fixed wheel gate are the magnitude of the loading, the size of the gate and the capacity for self or forced closure.

From the considerations of operational factors, it is obvious, that the gates may be subjected to full hydrostatic head plus vacuum head on account of sudden closure of the gate and downpull. Air vent is provided immediately downstream of the gate in the

top of intake transition to supply air to reduce the vacuum head. The design and function of air vent is discussed elsewhere. In case of Rihand Dam, even with the air vent, the gates are subjected to a vacuum head of 4.6 ft., and adopting a factor of safety of 2.5, a vacuum head of 11.5 ft. has been adopted for designs. The total head of water for which the gates have been designed works out to 160 ft.

During emergency closures the upstream face and the top of the gate are subjected to nearly constant pressure while there is a decrease in pressure on the downstream face and bottom of the gate on account of the conversion of pressure head into velocity head. This causes an unbalance of vertical pressures resulting in a downward force on the gate generally referred to as downpull. The downpull is reduced by providing suitable shape to gate lip and by providing recess in the dam face on the top of the intake opening. Downpull is computed on basis of tests made for similar installations on other dams or on basis of theoretical considerations[2]. It was computed to be about 56,000 lbs. for Rihand Dam gates for condition of restricted discharge. Downpull is to be taken care of in providing the hoist capacity.

The gate is designed so that it may close under self weight if possible. The forces opposing downward motion are friction of the wheels and the seal. Two types of bearings, viz., self-lubricating bearings and anti-friction bearings are available for use with wheels. The friction of the latter is about 1/10th that of the former. The frictional resistance of wheels is determined by the following formula.

$$F = \frac{P}{R}(f_r + rf_a)$$

where F = frictional forces in lbs., P = total water pressure against gate in lbs, R = radius of wheel in ft, r = radius of wheel pin in ft, f_r = coefficient of rolling friction = .003 for self lubricating or antifriction bearing, f_a = coefficient of axle friction = 0.1 for self lubricating bearing and 0.01 for antifriction bearing.

The friction resistance on account of seals is more indeterminate depending on the length of seal, contact area and deformation etc. It has been found that a doublestem rubber seal with brass bound nose as shown in the drawing is most efficient for such installations. The frictional force is given as follows.

$$F_s = .217\ H_a\ UL$$

Where F_s = seal friction, H_a = average hydrostatic head in ft, L = total length of seal in inches and U = coefficient of friction = .3.

The gate consists primarily of a skin plate supported by horizontal beams and vertical stiffeners which are supported by two vertical girders at the side ends. Fixed wheels are located between the vertical girders. Vertical girders are also attached to the middle of the skin plate to give it rigidity for lifting and for proper distribution of load. The lifting stem of the hoist is attached to three top horizontal beams. The wheels move on and transmit the load to the tracks located on the face of the concrete structure. The tracks are supported on trackbase which are embedded in concrete. The wheel load is transmitted to the concrete through the tracks and trackbase. Guide shoes are located near the top and bottom on each side of the gate to keep the gate in position laterally.

Gates can be made of welded or rivetted construction. The girders can also be either of rolled shapes or may be of welded plate construction. From considerations of economy and availability of steel, a completely welded gate has been decided to be got manufactured.

On account of the size of the gate and transport difficulties, the gates shall be made in five units which shall be rivetted together at site. Each unit has two wheels on either side. The vertical girders of the various units shall be joined at the webs only so as to provide a semiflexible arrangement which ensures that even in case of some misalignment all the wheels are uniformly loaded. The structural arrangement of the gate adopted was arrived at after consideration of economy for various alternative arrangements.

The skin plate is designed as a flat plate supported continuously on four sides. A convenient way of evaluation of end moments is by Dr. Marcus' equation[2]. An important factor in the structural design is the estimation of the length of the skin plate acting as flange of the horizontal beams and the stiffeners[2]. The skin plate is subjected to two dimensional stresses while acting alone and also while acting as flange of the horizontal beams. The combined stresses in various combinations should be within safe limits according to Westergaard's criteria[2].

Structural steel conforming to I.S. 226-1950 or equivalent is proposed to be used for gates and the safe tensile stress is 18000 lbs per sq. inch with a corrosion allowance of 1/16″. Alternatively the corrosion can be taken care of by adopting lower stresses of 15000 p.s.i. in tension.

The fixed wheels of the gate serve as the support for the gate leaf and transfer the loads to the track and trackbase. The wheels are spaced equally. To avoid undue concentration of stresses in wheel rim and track which may be developed on account of deflection of gate under load, wheel and gate drift etc, the track is given a suitable curvature so as to produce a rise of about 1/32″ at the crown. The wheels and tracks shall be made of cast steel 180 to 220 BHN and 250 BHN respectively. The dimensions of the wheel are determined from the consideration that the critical stress on the projeced area should not exceed the factor $24.5 \times BHN - 2200$. In addition the contact stress as determined by Hertz's formula[2] should

be within safe limits. The wheels are mounted on eccentric pins to provide for alignment.

The selection of antifriction bearings is made from the standard products of reputed manufactures, such that they are suitable for the conditions of operation and load capacity.

Disc springs are provided on either side of the wheels to centre them when they are off-load. Double flat springs may also be used.

The gate frames consist of track, track base and seal seats. During the first stage, concrete blockouts are left to house the trackbase and sufficient number of anchor bolts are embedded to provide the necessary bond between the stages of concrete and to provide means of fastening and aligning the trackbase. The trackbase is connected to these anchor bolts aligned and then the blockout is grouted. The track is connected to trackbase by removable cleats.

The design of track and trackbase is done according to Biots treatment of bending of infinite beams on elastic foundation. It is checked that the diamensions of track and trackbase are sufficient to contain the prism of projection of contact elipse between wheel and track[2].

For sealing of the gate, double steam rubber seals with a brass bound nose as shown in the figure have been found to be most satisfactory. Agreement is made for admitting water under full head on the back of seals so that positive closure is obtained. The seal is connected to the gate with stainless steel clamps and bears against a stainless steel seat.

Two spring backed shoes are provided on each side of gate to engage on the guide provided on the concrete face to keep the gate in position laterally.

The total weight of five gates and gate frames for all the six intakes is 521 tons. The gates are being manufactured in India by M/s Texmaco in collaboration with M/s. V.O.E.S.T. of Austria, in accordance with the detailed drawings and specifications issued departmentally.

Penstock Gate Hoist

Fig. 4 shows the general installation of Penstock Gate Hoists. Each hoist is supported on a frame spanning the gate slot at the top of dam. The hoist consists of three steel cylinders of 20 in inner diameter bolted together at flanges in which oil under pressure operates a piston. The piston is connected to penstock gate through a set of stems connected to one another through pairs of clevises. The clevises form a hinged joint. Oil under pressure is supplied from a common oil tank through individual hoist power cabinets. The operation of the hoist is controlled from panels located adjacent to each hoist. The panel has three push buttons marked 'CLOSE', 'OPEN' and 'STOP'. The pressing of a push button starts and completes the full action after which the hoist automatically comes to a stop. For emergency closure of penstock gate an emergency switch is provided at the powerhouse control panel. During operation of this switch all other push buttons in the control circuit become inoperative till the closing cycle has been completed.

The penstock gate position is indicated on the power cabinet dial by means of sprocket and chain connection with the piston stem. A set of red and green light indicators are provided at the adjacent operating panel and at power house control panel for each hoist and indicate the full open and full close positions of the gates respectively. Both the lights remain on when the gate is in intermediate position.

The design load capacity of the hoist is computed from the load exerted by the penstock gate during emergency gate opening cycle. The hoisting load is sum of gate weight, weight of moving parts of hoist, downpull force, and friction of gate and moving parts of hoist[3]. The capacity for Rihand Dam intake gate hoists works out to 250,000 lbs. The period of operation of the hoist is fixed as under:

(i) Maximum closing time at any reservoir elevation upto 880.00 – 3 minutes.

(ii) Minimum closing time at any reservoir elevation upto 775.00—1 minute.

(iii) Maximum opening time at any reservoir elevation upto 880.00—30 minutes.

The lift of Rihand Dam penstock gate is 30 ft. 11¼ in. For gate closure in 1½ minutes this gives a speed of about 20 ft-8in. per minute. This speed is controlled by a throttle valve connecting the oil below the piston to the top of hoist.

The hoist is designed to operate at a pressure of 1000 p.s.i., which pressure is not exceeded in the worst operating condition.

The gate and moving parts of hoist, in full open or partial open positions, remain suspended from piston which is supported on oil under pressure and any leakage of hydraulic oil past the piston causes the gate to lower. To minimise this leakage the piston is provided at its top with Cheveron or U-type oil-tight packing in addition to the conventional piston rings which are provided primarily to enable to replace the oil-tight packing with the piston in top position by removing only upper cylinder head. The stem is also packed with Cheveron or U-type packing in addition to the water-tight packing where it runs through the lower cylinder head. At about 12 inches from the end of closing cycle the downward motion of the gate is damped so that the piston contacts the lower cylinder head at the end of its motion with practically no shock.

For dissembly, the piston is brought to the top position and is locked by a hand wheel operated latch housed in the upper cylinder head. The hoist is lifted and all oil is drained off before dissembly.

As mentioned earlier the oil for all the six hoists is stored in a central tank of sufficient capacity.

Each hoist is operated from control panel located adjacent to the hoist. Each control panel has a control cabinet housing an electric motor, oil pump, motor starter, hydraulic equipment, piping, push buttons, pressure switches, relays and miscellaneous equipment required to complete the equipment. Separate motors, and pumps are provided for all the hoists from considerations of sensitiveness and reliability of operation. It is desirable to keep the controls as near the hoist as practicable so that the true pressure variations occurring in the hoist cylinder may be conveyed to control circuit.

Fig. 4 illustrates the lay-out and operational sequence of the control for one gate hoist. The controls consist of elementary circuits, selection of which depends upon the type and nature of service required. The circuits are made automatic in action by introducing pressure switches which make or break the circuit in a specified range of pressure. The pressing of a push button actuates a series of these control circuits. During opening and closing cycles the circuits operate the throttle valve so as to connect the oil in the space below the piston to the space above it. The flow of oil through the throttle valve is controlled during the entire cycle so as to effect the completion of the cycle in a predetermined manner. During large periods when the penstock gate hangs in full open position, the leakage of hydraulic oil past the piston may lower the gate and when this exceeds 3 in. from top position, an automatic circuit restores the gate to its full open position by operating the motor and feeding oil under pressure to space below the piston. In case the circuit fails to function and the gate continues to drop down by further 3 inches, an alarm is sounded both at unit control panel and at control room in power house necessitating the operation of the gate in the opening cycle.

The throttle valve is designed from the consideration that $V \times A \times Cd = Q$

Where Q = Average rate of evacuation of oil from the hoist cylinder

Cd = Coefficient of discharge.

A = area of orifice

V = Velocity through the orifice.

The average rate of oil flow is computed from the time of closure of penstock gate and total oil displacement during closing cycle of gate.

At any instant during gate-closing, the oil pressure acting from below on piston in the hoist cylinder balances the total downward force in the gate. This oil pressure less friction loss in byepass pipe gives the hydraulic head available at the orifice for conversion into velocity. The area of throttle valve thus determined, is adjustable, and is fixed for a certain opening on the field by trial and error so as to close the gate in emergency in less than the prescribed time.

In the control unit the motor is a squirrel cage, non reversible one of 10 H.P. capacity with 1000 p.s.i. delivery pressure pump. The control valve is a four way solenoid operated valve of spool type. The pilot valve is a four way hydraulic pressure operated spool valve. In addition there are sequence valves, pressure switches, relays, light indicators, pressure gauges and alarms.

The penstock gate hoists are being manufactured by M/S V.O.E.S.T. of Austria at a cost of about Rs. 5.31 lacs according to detailed departmental drawings and specifications.

Stoplogs

Stoplogs are provided to close and isolate the intake conduits either in emergencies or temporarily for the purpose of inspection of the intake works and the penstocks if and when the penstock intake gate gets out of order and has to be taken out for repairs. The stoplogs also enable the inspection and maintenance of intake gates in open or closed position without taking them out. Only one set of stoplogs will suffice for all the intakes, but stoplog guides will have to be provided in each intake structure.

A set of stoplogs consists of a number of units required to seal off an intake completely from bottom to top of the opening. Stoplogs are operated under balanced water pressure by gantry crane travelling on top of the dam. In closed position, the stoplog units rest one above the other. The cross beams of the individual units transfer the load which is coming on it due to water pressure through wooden seals to the guides embedded in concrete on the face of the dam.

The stoplogs shall be placed on the face of the dam in front of the penstock intake gates. The general layout of intakes with stoplogs for Rihand Dam is shown in Fig. 1.

For Rihand Dam 22 units, 4 of 6′ height, 4 of 7′ height, 4 of 8′ height, four of 9′ height and 6 of 10′ height have been used. The structural arrangement and design of each unit is the same as that for penstock intake gate except that wheels have been omitted[4].

To have tight seal between sides, the pressure on wooden seals should be kept high. Since at higher level the water pressure decreases, it is necessary that in order to have same pressure intensity between wood seal and guide the area of contact between the two should be reduced. As a matter fact the contact area should vary from top to bottom but as it is not practicable the contact area has been changed at three elevations only.

The shape of guide in the flow area is kept similar to the curve existing in the concrete face and the width of guide is extended up to the clear opening. Above the flow area, the width of the guide is deter-

mined by bearing capacity of concrete for the pressure coming on it at that elevation.

During the first stage, concrete blockouts are left to house the frames. An adequate number of anchor bolts are embedded to provide the necessary bond between two stages of concrete as well as to provide means of fastening and aligning the frames.

The total weight of stoplogs and guides is 400 tons. They are being manufactured in the Irrigation Department Workshop at Bareilly, according to detailed specifications and drawings issued to them.

Trashracks

Trashracks have been provided upstream of the intake gate and stoplogs to protect turbines etc., from objectionably large debris. The criteria of location and arrangement adopted for trashracks is discussed elsewhere. The total area provided for the trashracks is determined from the consideration that the capitalised cost of trashracks and power lost due to head loss through them is minimum. From this consideration it was found that the gross area of the trashracks should be such that the velocity through them should be 1.13 ft. per second. Trashracks with a surface area of 2660 sq. ft. per intake have been provided.

The trashracks are provided in five bays and extend upto a height of 56′. In each bay four units of $11' \times 14'$ are located, placed one on the top of another.

The trashrack units consist of vertical bars spaced about 5″ centre to centre supported on horizontal members which in turn are supported on end channels. The units are located in grooves in the concrete structure. The spacing of vertical members is governed from the consideration that it should be smaller than the smallest opening through the runners of the turbine.

The trashracks are normally subjected to almost zero head, but due to clogging they may be subjected to considerable water head. In accordance with standard practice for similar installations the vertical bars are designed to fail under a head of 20′. To provide least damage consistent with safety the horizontal bars are designed to fail at a head slightly more than 20′, and the concrete structure supporting the trashrack units is designed to withstand a head of 30′.[5]

From consideration of rigidity, the vertical members are kept a minimum of $\frac{1}{2}''$ thick and 2″ deep. The spacing of horizontal supporting members, and section of horizontal and vertical members is arrived at after trial and error from economy consideration. For Rihand Dam the vertical bars are $\frac{1}{2}'' \times 3''$, spaced 5″ c/c. giving a net opening of $4\frac{1}{2}''$. They are supported on horizontal flats $\frac{3}{4}''$ by 12″ placed 2′ $9\frac{1}{4}''$ c/c. The total weight of trashrack units for all the six intakes is 233 tons. The trashracks are being manufactured in Irrigation Department Workshop Bareilly, according to detailed drawings and specifications issued to them.

Intake Gantry Crane

An electrically operated outdoor travelling type gantry crane will be provided for handling the penstock intake gates and hoists for installation and dismantlinig for repairs and for the operation of stoplogs, and sluice bulkhead gates. The crane will be located and operated from the top of the dam. The runway rails are located 18′ centre to centre on the roadway and the track will be laid from the cranehouse on the right abutment to the spillway section of the dam.

The crane will be provided with 100 tons fixed block for handling the penstock intake gate and hoist and a 35 ton trolley hoist for handling the rest of the equipment. The size of the crane will be fixed so that it can handle the maximum size of equipment and so that the vehicular traffic may pass under the lowest cross arm.

The intake gantry crane is being procured through global tenders according to detailed specifications and drawings issued by the Department.

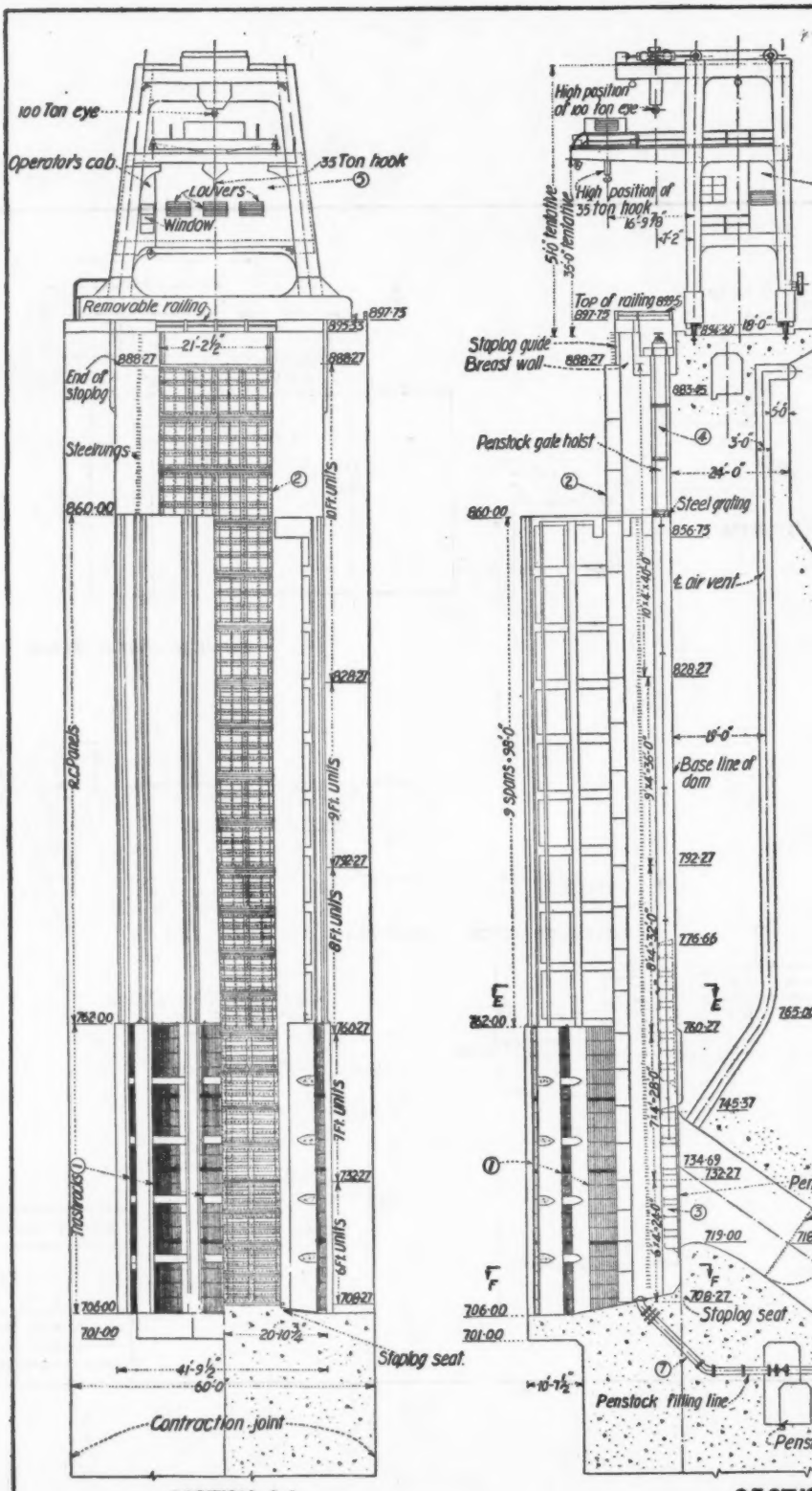
EQUIPMENT FOR SPILLWAYS

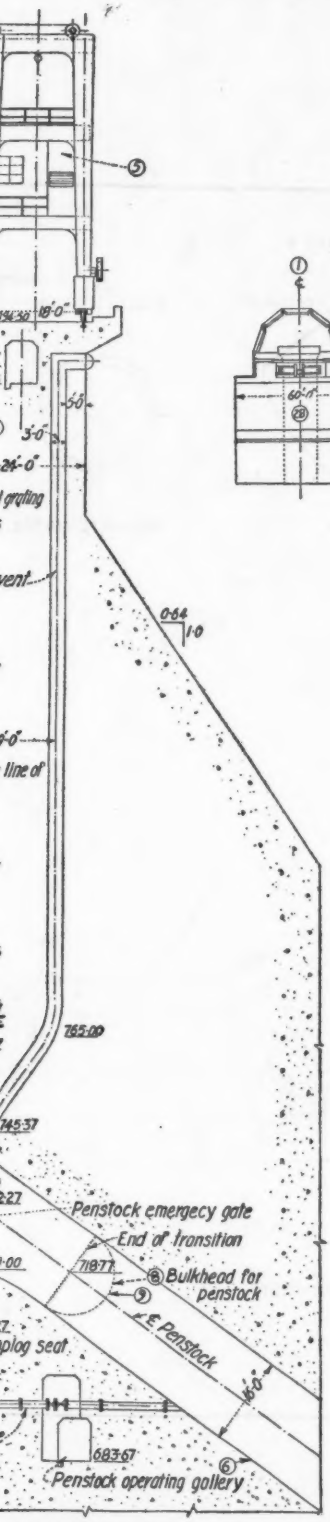
General Layout

The most economical and suitable arrangement to provide the spillway capacity as required from hydrological considerations consists of providing an over-flow spillway in the blocks 34 to 47, in a length of 520 ft. with crest elevation at 852.00. The choice of gates rested on Tainter gates as this type of gate is most economical and suitable for a controlled spillway because of its simplicity, light weight and low hoist capacity requirements. Since for a given discharge capacity, the total spillway cost varies only slightly with normal gate dimensions, the individual gate size was fixed to correspond to the practical and economical size of gates to suit the commonly used sizes of monolith blocks of the dams. It was, therefore, decided that the spillway shall consist of 13 bays of 40′ clear waterway with crest elevation at 852 while the maximum reservoir elevation had been kept as 880. The spillway is capable of routing the designed flood, even if it occurs when the reservoir is already full, in which case the water surface will rise to elevation 886, which is the maximum allowable high flood level. The layout and the salient features of gate and hoist are shown in Figs. 5 & 6.

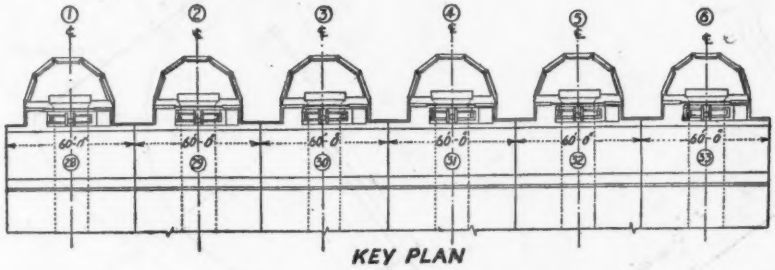
Spillway Tainter Gates

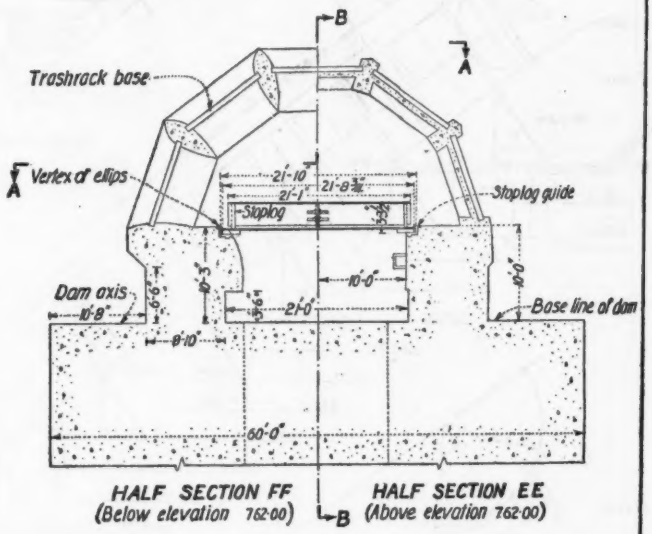
The installation and general features of the Tainter Gates used for Rihand Dam is shown in Fig. 5. The principal feature of Tainter gate is that the skin plate assembly is formed to a segment of a cylinder and it is supported through radial struts by


100 Ton eye
Operator's cab
35 Ton hook
Louvers
Window
Removable railing
897·75
875·33
21'-2½"
888·27
End of stoplog
Steelrungs
860·00
R.C. Panels
9 Ft. Units
828·27
792·27
776·66
752·00
760·27
745·37
734·69
732·27
719·00
708·27
706·00
701·00
20'-10¾"
41'-9½"
60'-0"
Contraction joint
Stoplog seat
SECTION AA
High position of 100 ton eye
High position of 35 ton hook
5'-0" tentative
35'-0" tentative
16'-9⅞"
4'-2"
Top of railing 889·5
894·30
18'-0"
Stoplog guide
Breast wall
883·45
5'-0"
3'-0"
Penstock gate hoist
24'-0"
Steel grating
856·75
t. air vent
9 spans @ 36'-0"
18'-0"
Base line of dam
Stoplog seat
10'-7½"
Penstock filling line
SECTI



SECTION BB

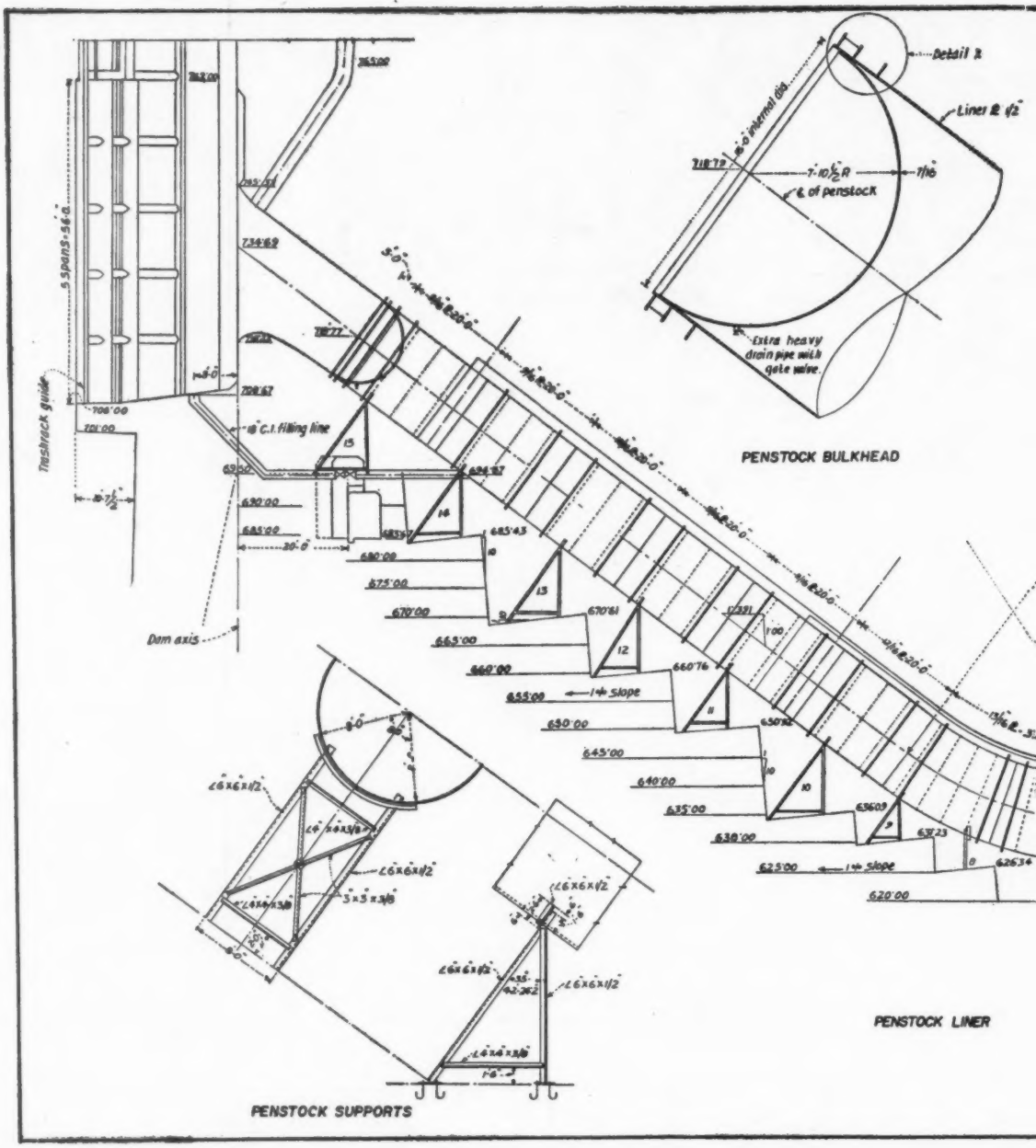




1. Trashracks.
2. Stoplogs.
3. Penstock gate.
4. Penstock gate hoist.
5. Intake gantry crane.
6. Penstock.
7. Penstock filling line.
8. Bulkhead for penstock.

FIG. 1

DESIGN & PLANNING DIRECTORATE RIHAND DAM		
PERMANENT MECHANICAL EQUIPMENT GENERAL LAY-OUT		
Designed Sd/S.P.H.S.C.S.	Checked. Sd/N.K.G.	Recmd. Sd/M.C.C.
Drawn. Sd/Tyagi	Subtd. Sd/S.P.B.	Appd. Sd/S.K.JAIN


Detail 'A'
Liner @ 1/2"
18'-0" internal dia.
T. 10 1/2" R
℄ of penstock
Extra heavy drain pipe with gate valve.
PENSTOCK BULKHEAD
5 spans @ 54'-0"
Trashrack guide
18" C.I. Filling line
Dam axis
1+ slope
PENSTOCK LINER
PENSTOCK SUPPORTS

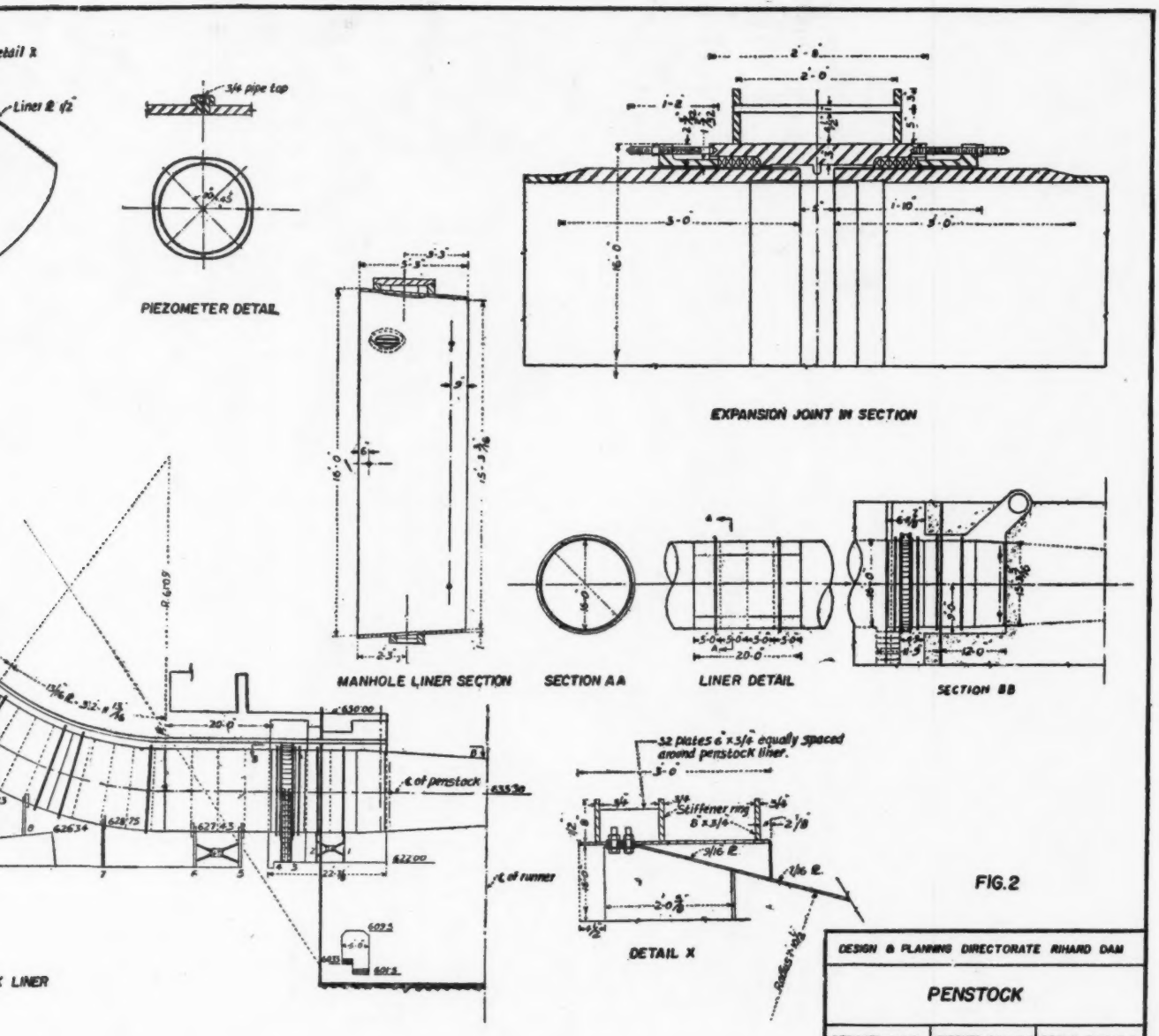
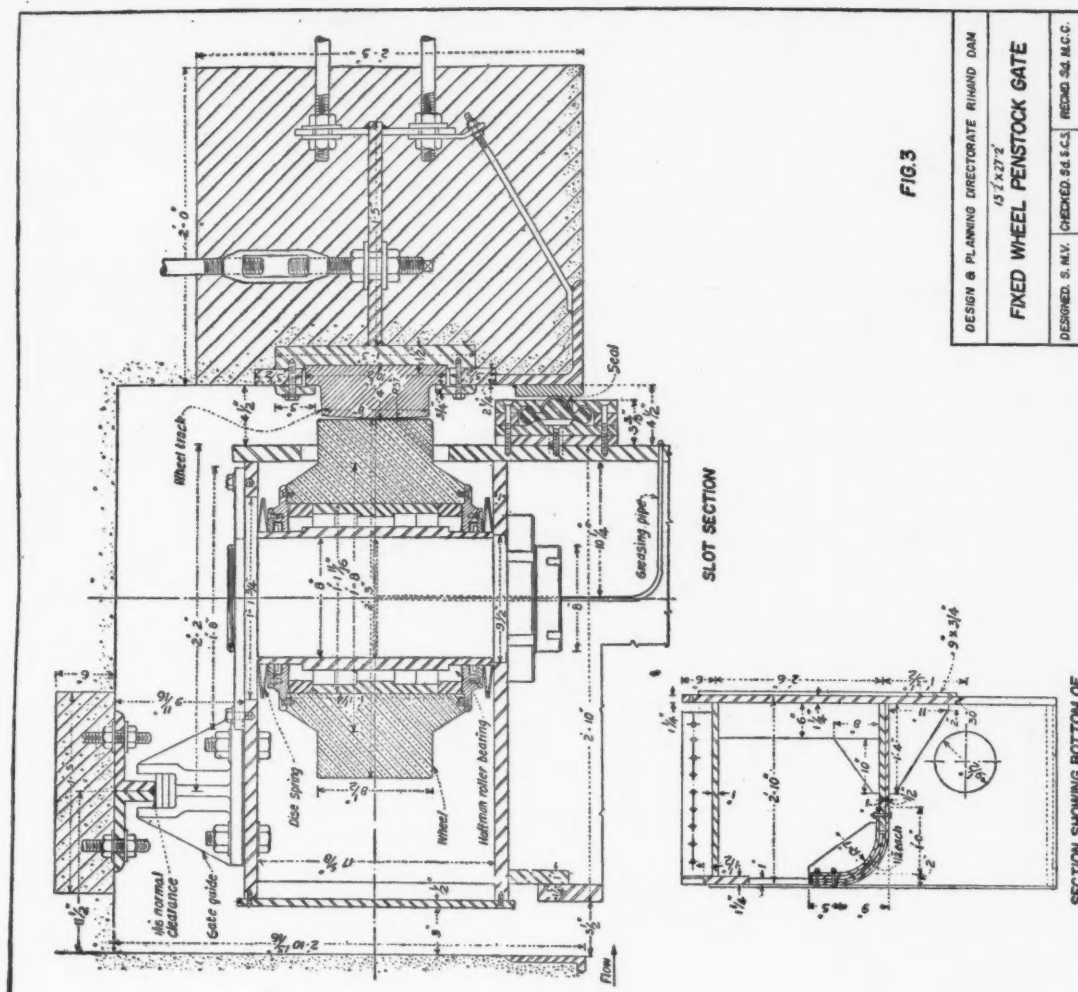


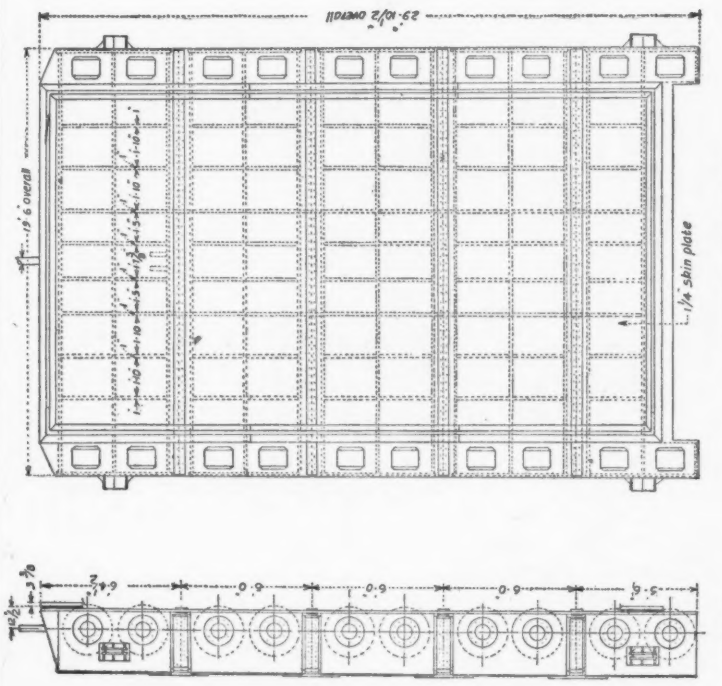
FIG.2

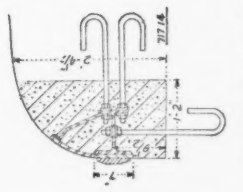
DESIGN & PLANNING DIRECTORATE RIHAND DAM		
PENSTOCK		
DESIGNED. N.K.G.	CHECKED. Sd. D.P.G.	RECMD Sd. M.C.C.
DRAWN J.S.Singhal	SUBTTD Sd. D.S.	APPD.Sd. S.K.Jain.

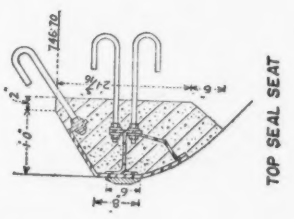
DESIGN & PLANNING DIRECTORATE RIHAND DAM			
13'1"x27'2" FIXED WHEEL PENSTOCK GATE			
DESIGNED. S. N.V.	CHECKED. Sd. S.C.S.	RECMD Sd. M.C.C.	
DRAWN J.S.Singhal	SUBTTD. S&D.S.	APPD. Sd. S.K.Jain	

FIG.3

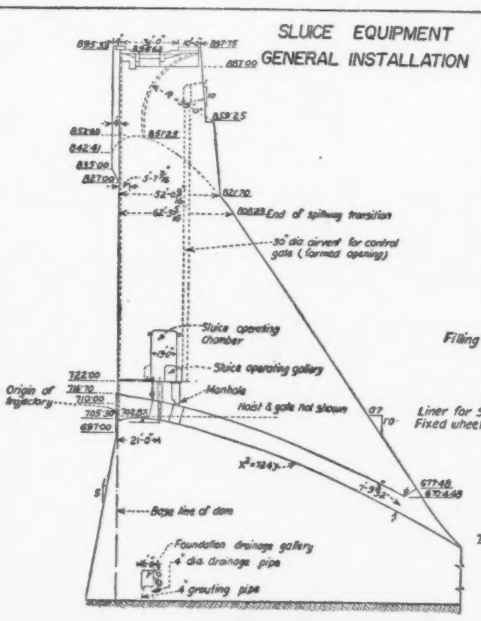




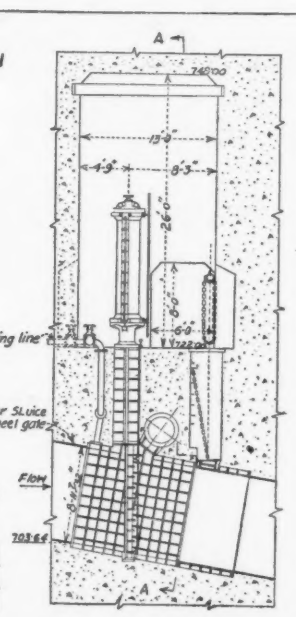

BOTTOM SEAL SEAT


TOP SEAL SEAT

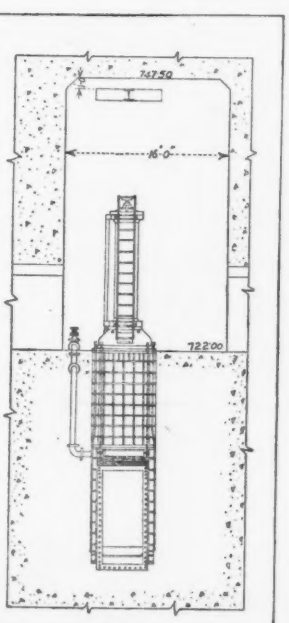
SLUICE EQUIPMENT
GENERAL INSTALLATION



TYPICAL SECTION THRO' SLUICE



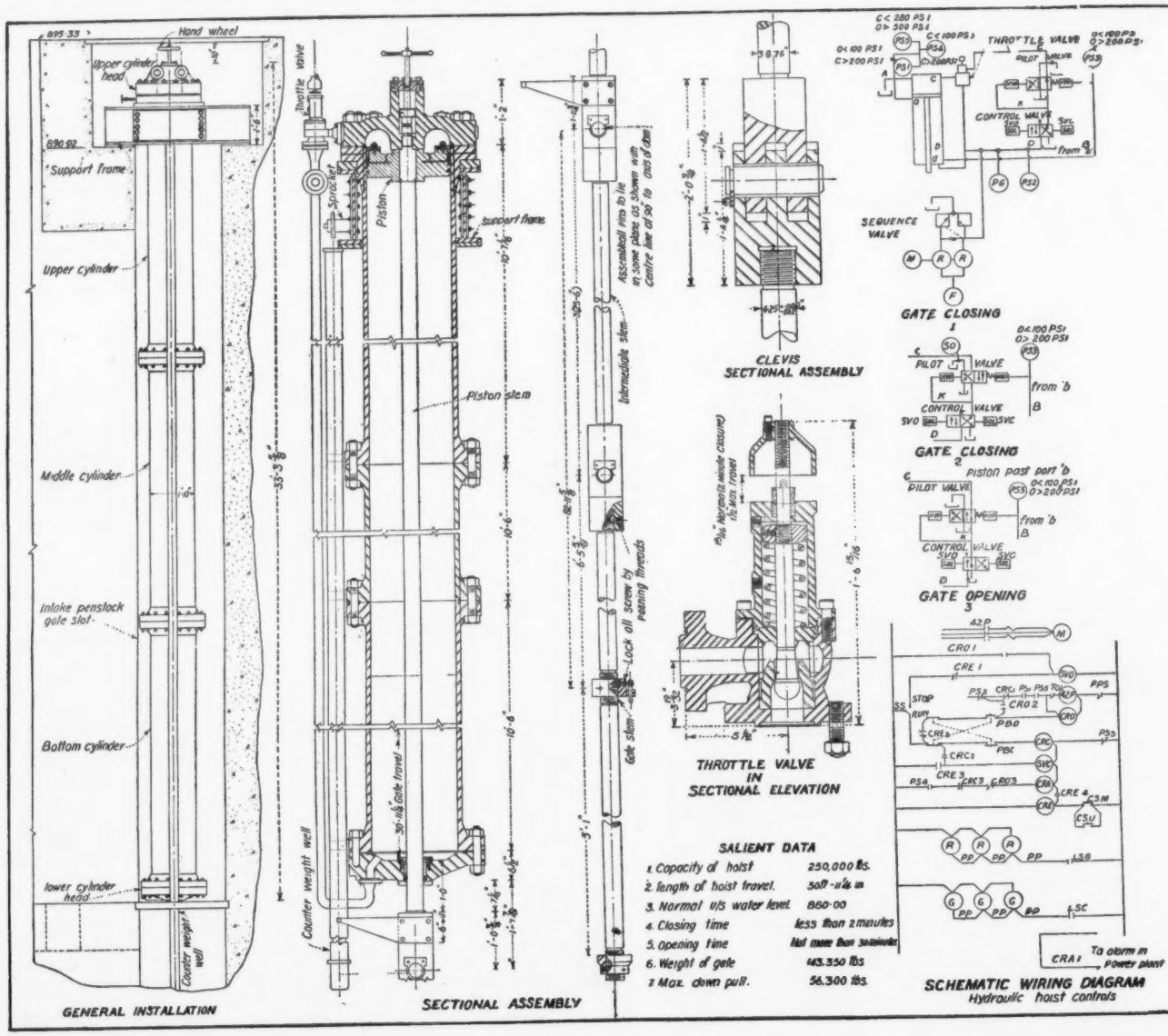
ELEVATION-PART IN SECTION



VIEW AA
(Airvent not shown)

DRAWN BY R.P.SANSHAL

FIG. 7



SALIENT DATA

1. Capacity of hoist	250,000 lbs.
2. length of hoist travel.	30ft -11½ in
3. Normal u/s water level.	860·00
4. Closing time	less than 2 minutes
5. Opening time	Not more than 3 minutes
6. Weight of gate	143.350 lbs
7. Max. down pull.	56.300 lbs.

CLOSING CYCLE

Piston near top of cylinder solenoid control valve in open position, red signal light on Put stop switch in run position. Closing cycle operated from gate panel or Power House control room

PUSH CLOSE BUTTON PBC

- Relay CRC and solenoid SVC energises. CRC_2 and CRC_3 open and prevent restoring cycle from starting and the alarm circuit from making
- Green light on, Red light on
- CRC drops out but PS_1 breaks restoring circuit Alarm circuit is open through CRC
- PS_3 opens and de energises CRC, SVC and breaks alarm circuit
- Red light turns off Green light remains on

Control valve shifts to close position, oil under piston escapes through line 'D' and 'C' and opens throttle valve and gate starts to close Piston passes tap 'a'

Piston nears close position and passes tap 'b' pressure drops off in line 'B'. Pilot valve spool shifts, and oil from under Piston passes to tank through line 'D' and arrive at Line C is connected to tank and throttle valve closes

Piston comes to rest, pressure drops and pilot valve returns to normal position

STOPPING & REVERSING.

The gate may be stopped at any position in either the opening or closing cycle

1. To stop during the closing cycle turn "stop" switch 33 to stop position.
2. To stop during opening cycle turn stop switch 33 to stop position.

OPENING CYCLE

Piston at bottom of cylinder, solenoid control valve in close position, green light on Put stop switch in run position. Opening cycle operated from gate panel only

PUSH OPEN BUTTON PBO

- Relay CRO and starter 42 P energises. CRO, makes and energises SVO CRO_3 opens and prevents CRA from energising
- Red lights turn on, Green lights on
- PS_3 closes
- PS_1 closes and makes restoring cycle circuit PS_4 opens and breaks alarm circuit
- PS_2 opens and de-energises relay CRO and starter 42 P motor stops.
- Green light turns off Red light remains on

Control valve shifts to open position connecting line 'C' to drain and blocking line 'D'. Pump starts & pressure built up under piston and gate starts to open.

Piston rises and covers tap 'b' Pressure in 'B' drops. Position of pilot valve remains unchanged.

Piston continues to rise and passes tap 'a'

Piston stops against cylinder head, pressure builds upto 800 lbs/

Gate remains in the open position with the piston supported on oil (see restoring system)

EMERGENCY CLOSURE

The emergency closing cycle initiated from C.S.M or C.S.U. takes preference over every thing including stop switch SS

NOTES :-

1. In case of Power failure solenoid control valve may be shifted manually to close gate
2. With oil pressure balanced in lines 'B' and K spring holds pilot valve in balanced position shown in figures 1 and 3.
3. Oil pressure in line 'B' alone tends to help the spring to keep pilot valve in balanced position
4. With oil pressure in line 'K' alone pilot valve position shown in figure 2 is obtained

NORMAL RELAY POSITION

CRC 1	closed
CRC 2	open
CRC 3	closed
CRO 1	open
CRO 2	open
CRO 3	closed
CRE 1	closed
CRE 2	closed
CRE 3	open
CRE 4	open
CRA 1	open

RESTORING SYSTEM

Due to Hydraulic leakage, the top of piston nears tap 'a' The pressure will drop in line 'A' As the pressure reaches 250 psi, PS_2 will close and motor contactor 42 P and opening relay CRO will make & seal in. Pump will start and piston will rise and stop against cylinder head Pressure will build upto 800 lbs/Sq.inch. PS_2 will open Starter 42 P and opening relay CRO will drop out and motor will stop

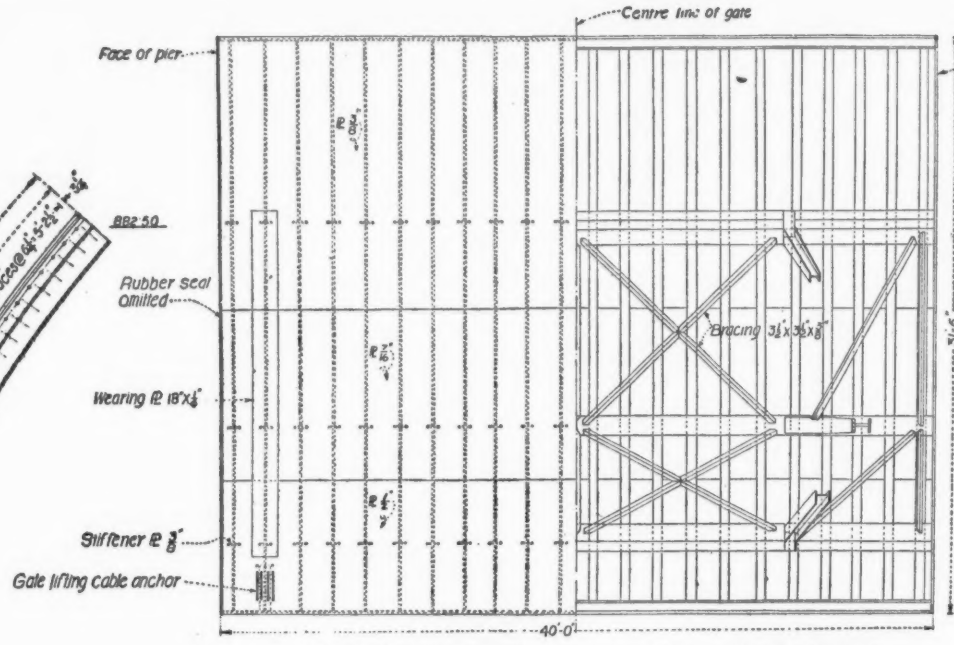
Should the restoring system fail, further drop of Pressure in line 'A' to 100 lbs/Sq.inch will make alarm circuit through PS4

PRESSURE SWITCH

- PS 1. pressure switch O< 100 lbs/□" C> 200 lbs/□"
- PS 2. pressure switch O> 800 lbs/□" C< 400 lbs/□"
- PS 3. Pressure switch O< 100 lbs/□" C > 200 lbs/□" (opens when gate nearly closed)
- PS 4. pressure switch O> 200 lbs/□" C< 100 lbs/□"
- PS 5. pressure switch O> 500 lbs/□" C< 280 lbs/□"

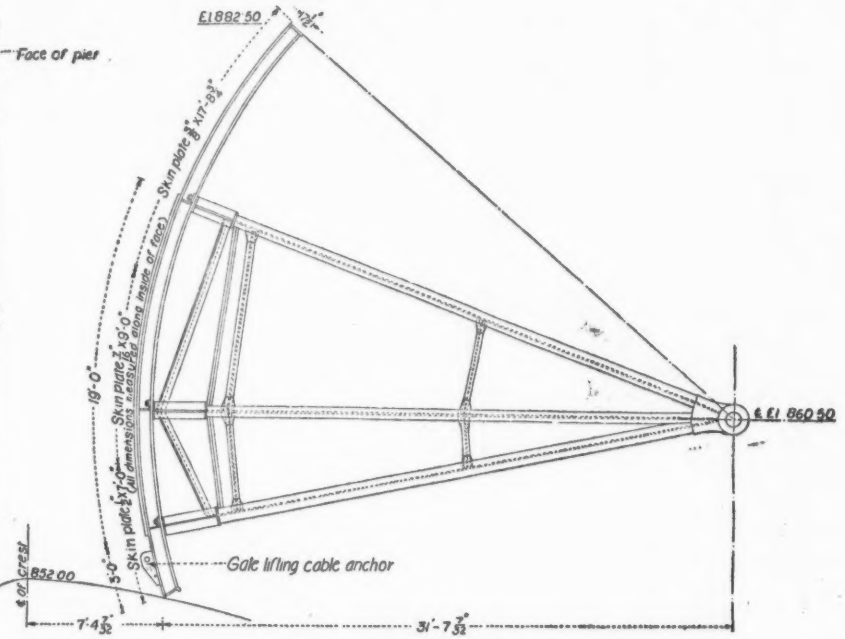
FIG. 4

DESIGN & PLANNING DIRECTORATE RIHAND DAM		
PENSTOCK GATE HOIST		
Designed SE SINHA	Checked Sdl/ANWARI	Rec.md Sdl/M.C.C.
Drawn Sdharma	Subtd Sdl/S.C.SINHA	Appd Sdl/S.N JAIN

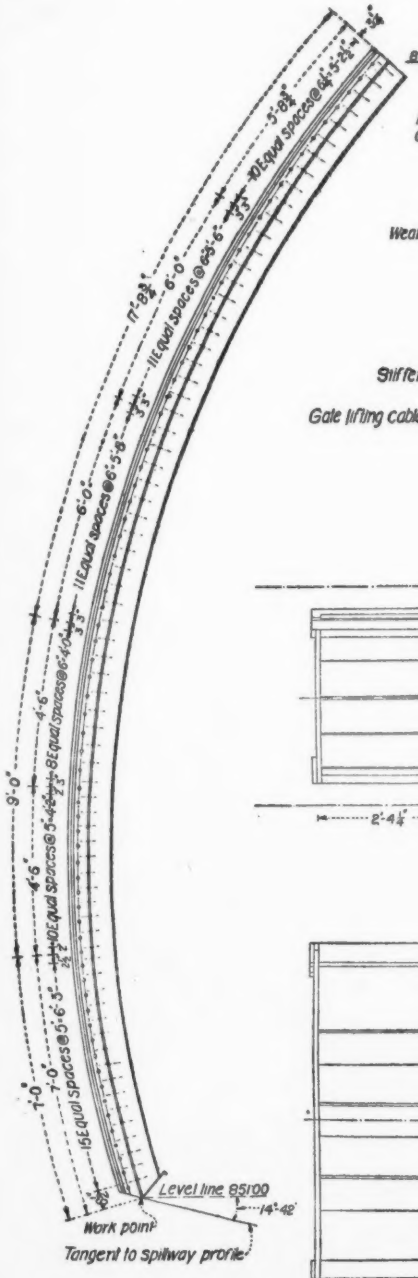


HALF U/S ELEVATION

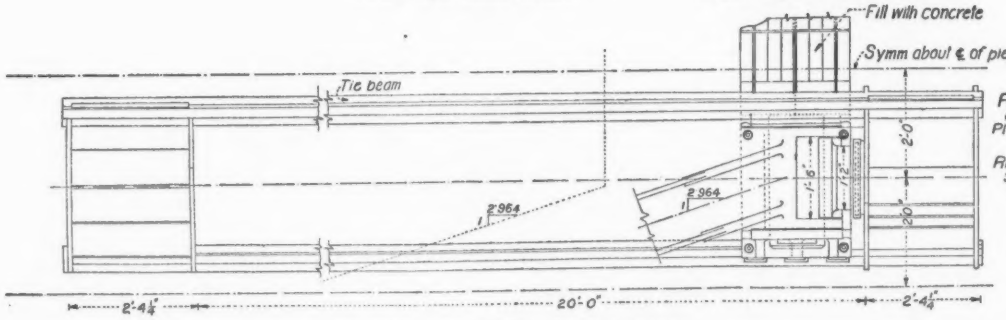
HALF D/S ELEVATION



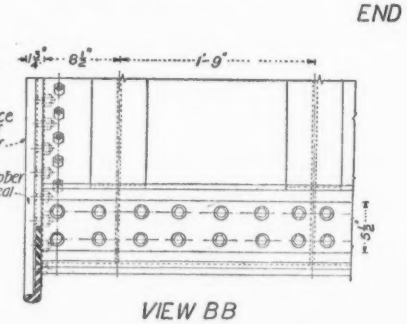
END ELEVATION



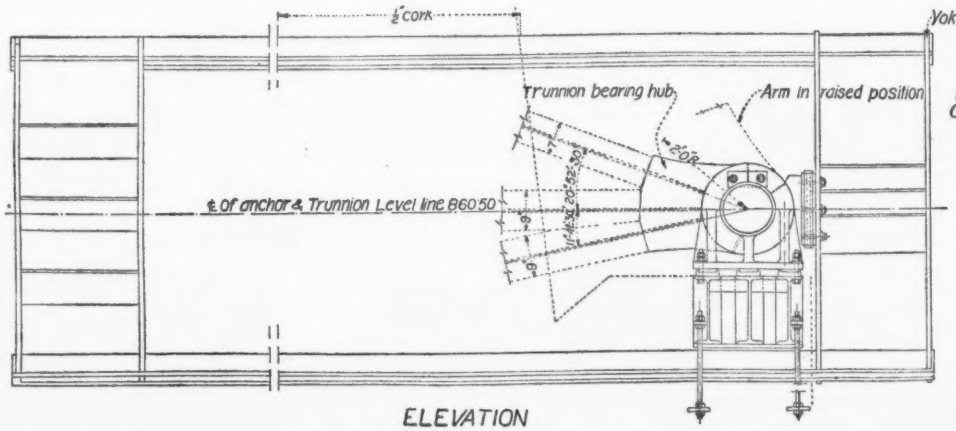
END VIEW (Side seal.)



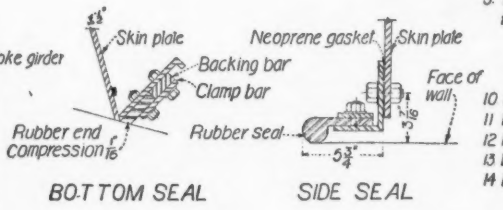
PLAN OF TRUNNION & ANCHORAGE



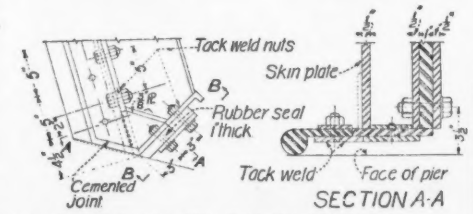
VIEW BB



ELEVATION



BOTTOM SEAL SIDE SEAL



SECTION A-A

ELEVATION OF SIDE & BOTTOM SEALS

SALIENT DATA

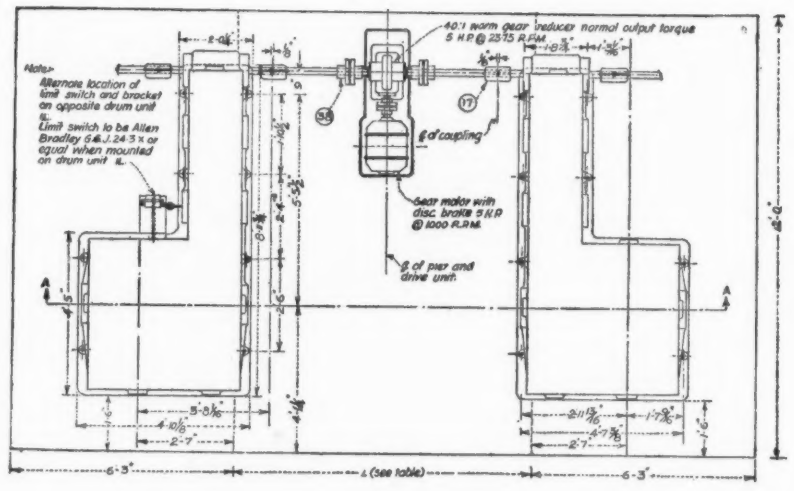
1 Maximum water surface elevation in reservoir	El 880.00
2. Crest of the spillway	El. 852.00
3. Sill of the gate	El 851.00
4. Top of gate	El. 882.50
5. No of gates	13
6. Size of gate	40'X31.5
7. Gate bottom in open position	El 878.59
8. Length of skin plate along curve	33'-6⅜"
9. The thickness of the skin plate is as follows for various heights	
(i) top to 15.5'	⅜" thick
(ii) 15.5' to 24.5'	½" "
(iii) 24.5 to bottom	⅝" "
10 Weight of gate	73,550 lbs
11 Weight of embedded parts.	6,500 "
12 Weight of anchorages	17,000 "
13 Weight of complete one assembly	97,050 "
14 Hoisting capacity required for one gate	88,000 "

FIG.5

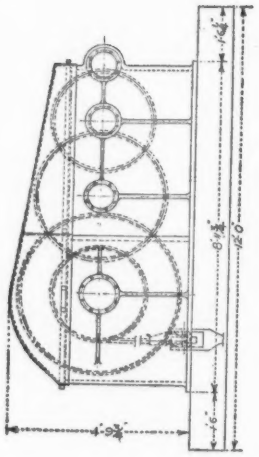
DESIGN & PLANNING DIRECTORATE RIHAND DAM

40'X31.5' SPILLWAY TAINTER GATE

DESIGNED:- R.N Verma.	CHECKED:- Sd/N.K.Gupta	RECMD: Sd/M.C.Chaturvedi
DRAWN:- D.C Choudhury	SUBTTD:- Sd/R.N.Verma	APPD:- Sd/ S.K.Jain.

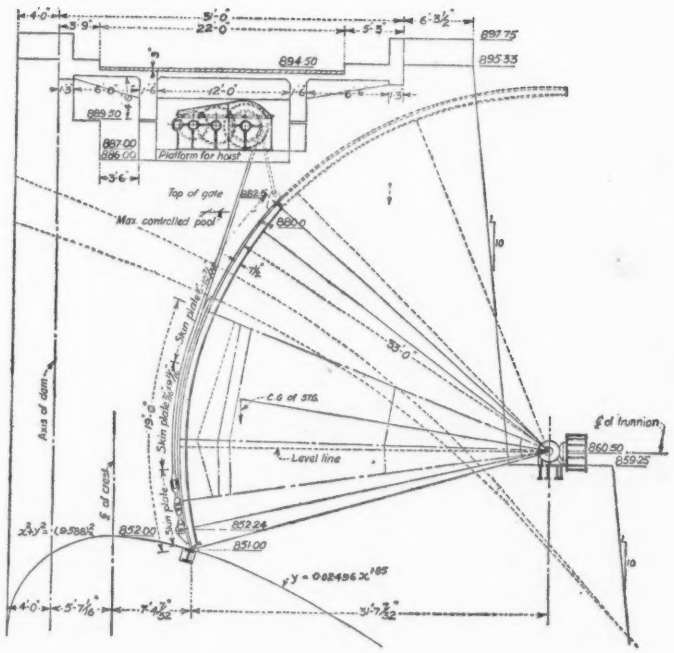

40:1 worm gear reducer normal output torque
5 H.P @ 2575 R.P.M.
Note:-
Alternate location of limit switch and bracket on opposite drum unit lL.
Limit switch to be Allen Bradley G.E.J. 24 3 X or equal when mounted on drum unit lL.
℄ of coupling
Gear motor with disc. brake 5 H.P @ 1000 R.P.M.
℄ of pier and drive unit.
L (see table)

PLAN

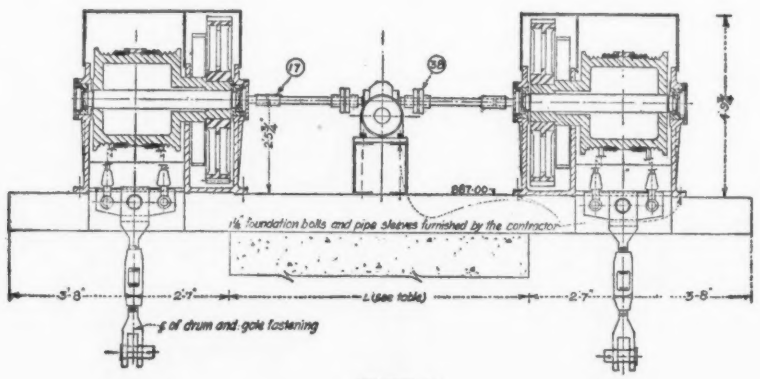


END VIEW

NO OF BLOCK		WIDTH OF PIER
FROM	TO	"L"
-	34	6'-0"
35	37	8'-0"
-	38	12'-0"
39	42	8'-0"
-	43	12'-0"
44	46	8'-0"
-	47	6'-0"



END VIEW



SECTION AA

1. Input rating 5 H.P. at 1000 R.P.M.
2. Maximum lift 30 feet.
3. Lifting speed = 0.69 ft per minute, at 1000 R.P.M input.
4. Total reduction 8320:1.
5. Speed of drum shaft 0.1147 R.P.M.
6. Hoist to be shop assembled for inspection.
7. Centre lines of rope connections on drums to be in alignment when couplings 17 and 38 are assembled.
8. Lengths of drive shafts to suit motor, worm gear unit and controlling dimensions on the drawing.
9. The contractor shall furnish the details of structural steel pedestal for drive unit.

FIG. 6

DESIGN & PLANNING DIRECTORATE RIHAND DAM		
SPILLWAY TAINTER GATE HOIST CAPACITY 85,000 POUNDS GENERAL INSTALLATION		
DESIGNED. Sd S.C. SINHA	CHECKED Sd. N.K. GUPTA	RECMD. Sd. M.C.C.
DRAWN. R.M. AGNIHOTRI	SUBMTD. Sd S.C. SINHA	APPVD. Sd S.K. JAIN

DESIGN & PLANNING DIRECTORATE IRRIGATION DAM		
4'x9' SLUICE GATE AND HOIST SECTION ASSEMBLY		
DESIGNED SRA & CHECKED SA-MRA.		RECMD SA-MLC
DRAWN RPSHARMA	SUBTD SP.S.B.	APPD SA-SKA

FIG. B

SECTION THROUGH GATE SLOT

SECTION FF

DETAIL R

SECTION AT LOCAL INDICATOR

PRESSURE PIPE CONNECTION AT CYLINDER HEAD

HANDWHEEL

HANGER ASSEMBLED

HANGER ENGAGED

CYLINDER HEAD

PISTON STEM

PISTON

CYLINDER

BONNET COVER

BONNET

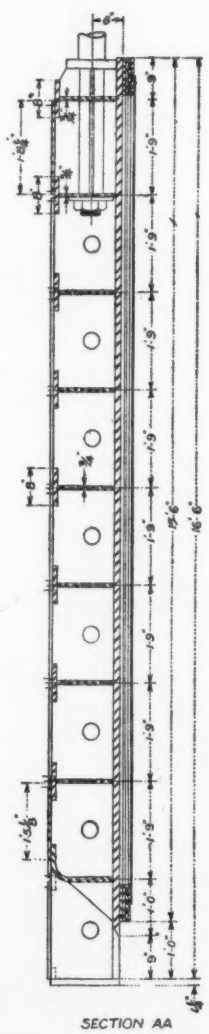
ELEVATION 72200

GATE LEAF

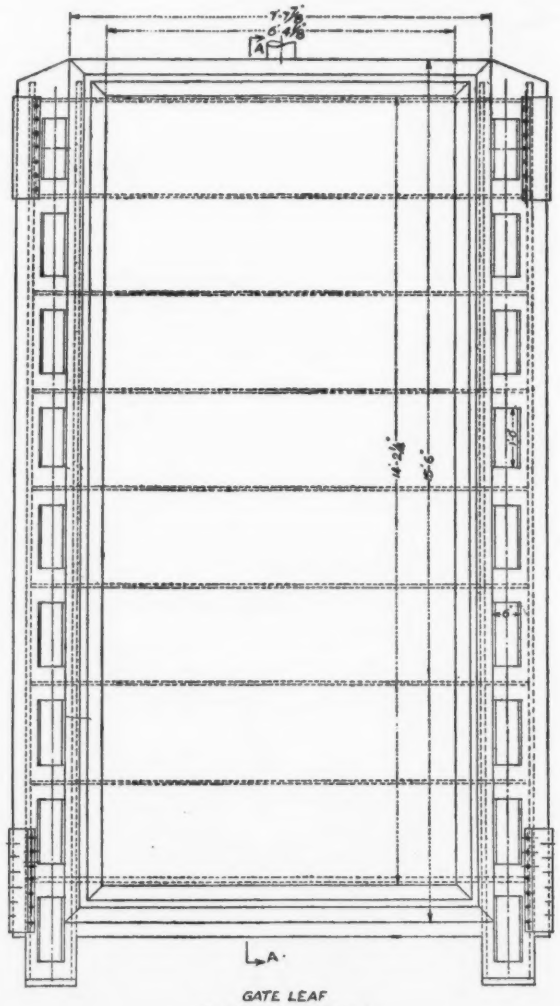
GATE FRAME

SEAL SEAT BASE

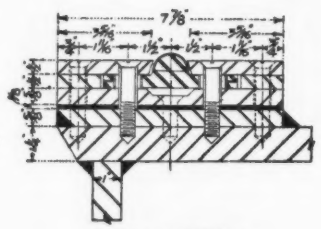
THESE SURFACES TO BE FLUSH AT ASSEMBLY



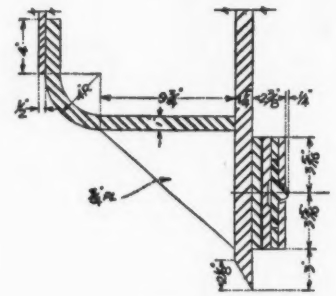
SECTION AA



GATE LEAF
DOWNSTREAM ELEVATION



SEAL ASSEMBLY.



GATE LIP

GATE AND SLOT SECTION

FIG. 9

DESIGN AND PLANNING DIRECTORATE, RIHAND DAM		
6·67' X 14·23' SLUICE FIXED WHEEL GATE		
DESIGNED Sd/-D. SINGH	CHECKED Sd/-M.K. GUPTA	RECMD. Sd/- P.C. GUPTA
DRAWN R.N. ASHWATHI	SUBMTD. Sd/- D. SINGH	APPRD. Sd/- KRISHNA MURARI

pins anchored in the piers which carry the entire thrust of the water load. The skin plate is made concentric to this pin so that the hoisting load consists, essentially, of the weight of the gate only. In principle, the standard arrangement has been adopted but by making several studies to determine the most economical arrangement a saving of about 18.5% compared to the standard design of the moving parts for same size and similar load and allowable stress conditions was made. The anchorage design adopted also produces a lot of saving compared to the conventional design[6]. The size of the gate is $40' \times 33'\ 8\frac{3}{4}''$. The weight of the moving parts for each gate is 73,553 lbs. The weight of anchorages, wall plates and embedments is 26.053 lbs, so that the total weight of the gate is 99,606 lbs and the weight of the moving parts per sq. ft. is 54.5 lbs.

The arrangement adopted consists of a skin plate stiffened and supported by row of vertical beams continuously welded to the skin plate. The thickness of the skin plate varies from 3/8″ at the top to ½″ at the bottom increasing in increments of 1/16″. The vertical members consist of $7\frac{1}{2}'' \times 6''$ T section @ 22.5 bs per foot, placed 22″ centre to centre. The face structure is supported by 3 horizontal girders so spaced as to produce equal bending moment at the critical sections of the vertical members and also to have equal loads at them as far as possible. The horizontal girders are supported on inclined struts at each end. The strute are inclined sufficiently in the horizontal plane so that the distance from point where struts intersect the horizontal beams to the near end of the horizontal beam is 1/5th of the span of the beam. The struts are inclined in this manner to balance positive and negative bending moment in the horizontal beams and to avoid carry over of moments in the structure. The struts are connected to a cast iron trunnion hub. The trunnion hub is engaged with a trunnion shoe with a forged steel pin suitably provided with a lubricated brass bushing. The shoes are mounted on independent plates which through a set of tension beams, are in turn connected to independent anchorage blocks embedded well within the pier to which the hydrostatic thurst is ultimately transferred. The lateral thrust is taken up by a thrust block provided adjoining the tension assembly. Tie girder was avoided for reasons of providing suitable clearance.

The arrangement adopted differs from the conventional arrangement in having independent plates for shoes instead of a common trunnion girder and in locating the plates such that load is transferred axially to the tension bars. This avoids subjecting the anchorage assembley unnecessary cantilever moments, or extra loads in case when one gate is open and the other is closed. The arrangement adopted is more efficient and considerably economical than the conventional arrangement.

The location of the gates is governed by the consideration that the trunnion should be above the floodnappe, the thrust should be transmitted nearly horizontally, the gates should clear the top bridge and hoist equipment placed below the bridge and that the whole arrangement should be economical. After several trials the arrangement shown in the figure was arrived at. The sill level of the gates is kept at 851.00 the crest level being 852.00. The trunnion level is kept at 860.50 and the skin plate has a radius of 33′. The top of the gates in the close position is kept at 882.50 to prevent wave wash even when the reservoir is completely full.

The gates are designed for hydrostatic load in normal conditions. In emergency conditions the gates are designed, in addition, for the wave pressure caused on account of waves in the long fetch of the lake. The wave pressure is computed by Sainflou's method[7] which worked out to 6′ uniform water head. The skin plates are provided with a corrosion allowance of 1/16″. The steel conforming to I.S. 226-1950 shall be used for the stage frames and anchorages.

Important factors of the structural design[7] are, determination of skin plate width acting as flange of vertical members, determination of combined stresses, and evaluation of loads for various conditions of water level and gate openings. The bending moments in the horizontal girders and supporting struts have been computed for both the conditions of hinged and fixed support and the members have been designed for the worst case. Effect of chain loads has been taken in account.

It is sometimes possible that one of the supporting ropes may break and gate may be subjected to torsional load or that jamming may take place in the trunnion pins and the struts may also be subjected to torsional loads. Suitable bracings have been provided in the face assembly and in the strut assembly for these contingencies.

Sealing of the gate is done by providing music note type of seals on the sides which travel on stainless steel bearing plates provided on the face of the piers. At the bottom of the gate a rubber seal is provided which rests on a stainless steel bearing plate. Suitable anchorages and levelling devices are provided for all the bearing plates to fasten them to first stage concrete and to align them. All the seals are also adjustable.

Three rollers are provided on each side of each gate to guide the travel when the gate is being raised. The rollers are adjustable and travel on suitable wall plates.

Total weight of all the thirteen gates is 580 tons. The gates shall be got made within the country according to detailed departmental drawings and specifications. Tenders had been invited on the basis of the detailed drawings and are at the moment under study for award of contract.

Spillway Tainter Gate Hoist

For bigger tainter gates requiring a hoist capacity exceeding 40,000 lbs, drum hoists are generally used. The hoisting arrangement and its location has been determined from the consideration of the size of gate, operating criteria of reservoir, space available for its installation, and presentation of a neat appearance on the upstream side of the dam. The general arrangement is shown in Fig. 6.

The drive unit consists of a low horse power induction motor or a commercial gear motor with a disc brake, directly connected to a commercial worm gear reducer on a common base plate. The drive shaft from the worm gear is connected to two rope drums on either side of the gate through suitable speed reducers. The speed reducers are of spur gear type and have been preferred to helical or herring bone type from consideration of economy as well as availability within the country. The drive shaft is suitably supported on pillow blocks so as to obtain uniform lifting of gate at both the ends.

The hoist drum assembly shall be located on projections cantilevered from piers at El. 887.00. For each span the hoist drum assemblies are connected by common drive shaft which also connects them through flexible couplings to the drive units at the two piers.

The gate is lifted through two ropes at either end of the gate, connected to the gate at one end and to the hoisting drums at the other end. The ropes are located upstream of the gates. The arrangement adopted has been preferred to use of chain instead of ropes or use of chain sprocket downstream of the gate as these involved considerable extra cost without providing comparative advantage.

A limit switch shall be provided on the hoist drum assemblies for each spillway tainter gate. The purpose of the limit switch shall be to positively stop the driving motor and the hoisting of the gate as soon as the end of the gate travel is reached. For further safety, gate stops are located in the piers which preclude lifting of gate beyond a certain opening.

The drive unit on an intermediate pier can operate either of the two gates located on the two sides of the pier depending on the connection through the flexible coupling. Therefore only one of the flexible couplings on either side of the drive unit shall be connected. For normal operation this arrangement shall provide one drive unit for each gate. In an emergency, however when a drive unit goes out of order, the drive unit on the other pier can operate the gate by altering the flexible coupling connections.

The drive unit on each pier shall be actuated by remote operation from a control board located in switch board room in Passenger Elevator Tower in block 34 of the dam as well as from the control room of the power house.

The operation of the gates is arranged in such a way as to open the gate in increments of one foot from full closed position to half open position beyond which the gate can be opened to full open position in a single continuous operation. The operating panel on the control board has three push buttons marked, 'open' 'close' and 'stop' through which the operation is governed. The gate opening indication at the control board panels is provided by means of synchronised selsyn transmitter and receiver set which indicates the gate opening of all the spillway gates separately.

The hoist has been located under the bridge and suitable hatches on the roadway are provided for taking out the hoist for repairs.

The hoist capacity is estimated by summing up the loads at the hoist rope due to weight of gate, trumnion friction, seal friction and weight of rope. The design capacity has been based on the maximum load at normal speed taking the transmission losses into account loads on compoment parts include the above load and the applicable friction losses in the reducers, bearings, gear trains and rope. The normal hoisting load distribution at the two points of the hoisting are 60% and 40% of the total hoisting force. Shock, impact and wear factors are considered negligible and have been disregarded. The hoist speed has been varied so that the horse power requirement approximately matches a standard motor's rating. From this consideration a hoisting capacity of 85,000 lbs with a hoisting rate of about one foot per minute has been found to be sufficient.

The electrical equipment provided on the hoist consists of hoisting motor, electrically operated brakes and controls. The hoisting motor is squirrel cage induction motor of high torque, high slip type with a frame of splash proof construction. Motors are of continuous rating in order to obtain the lowest overall cost for electrical system. The brake is internally mounted disc type or the shoe type, spring set electrically released with a capacity capable of overcoming the running torque of the motor.

The controls, consist of controller, limit switch, selsyn indicators and master control panel. The controller is of the combination air circuit breaker disconnect and reversing magnetic contactor type with thermal overload protection, enclosed in water tight and dust tight enclosing case. Two disconnects are required to allow connections to be made to either of the two dam feeders. A main line contactor is provided to give additional reliability and safety. The overtravel contacts of the limit switch are connected in the circuit of the main line contactor.

The limit switch is of the heavy duty type and of precision construction in order to insure reliable operation of the control system.

The hoists are proposed to be got manufactured in India. Tenders have been invited on basis of de-

tailed departmental drawings and specifications. Tenders have been received and are under study for award of contract.

EQUIPMENT FOR OUTLETS

General Layout

For each of the two outlets the control equipment consists of a hydraulic hoist operated slide gate installed at a distance of 21 ft. from dam axis with provision to use a fixed wheel gate at the upstream face of sluice. Only one fixed wheel gate is provided for both the sluices and shall be used for inspection and repair of slide gate and the conduit upto it. The general installation of sluice equipment shown in fig. 7, while the details of gate and hoist is shown in fig. 8.

For the range of head encountered on Rihand Dam slide gates are found to operate satisfactorily both for full and partial opening[8]. The other types of gate that can also be used are, jet flow and ring follower gates which are however, more complicated and costly. Tainter gates, which though efficient and economical are rather new in the field and for the limited scope of use in the sluice operation of Rihand Dam did not offer appreciable saving. Valves may also be used but their use is unnecessarily complicated and costly. After analysing the various possible arrangements of equipment it was decided to use two slide gates arranged in tandem. Slide gates besides being economical, simple, and efficient in operation, can be made in India, whereas most of the other type of gates will have to be imported.

The hydraulic hoist for the slide gate is located in a chamber directly above the gates. The gate is located in such a manner that the distance of the upstream wall of the gate operating chamber from the upstream face of the dam is approximately 10% of the head at the gate chamber. Normally the slide gate shall be used to close or open the sluice. In emergency when the service gate is under repairs, the sluice fixed wheel gate described later will be used.

Slide Gates

A slide gate consists of two rectangular heavily ribbed structures which form the water passage upon the upstream and downstream sides of the leaf for a longitudinal distances approximately equal to that of the horizontal width of this passage. These two structures known as the gate frames, are bolted together where their adjacent mating vertical flanged faces are brought into water-tight engagement. These frames are arranged to form a vertical slot or recess in the walls on each side of the water passage, within which the gate leaf is received, suppported, and guided in its opening and closing movements. The gate consists of upstream skin plate supported on a system of horizontal beams and stiffeners designed to transfer the waterhead on the skinplate to the vertical ends of the gate. The leaf is usually provided with bronze seal bars on its downstream face which rest, when the gate is closed, upon similar bronze seat bars of a different composition or upon monel seat bars attached to the downstream sides of these slot walls. The full water load against the upstream face of the leaf is carried to its seal bars and transferred into the seat bars on the gate frames, and in consequence whenever the leaf is moved upward or downward, the leaf seal bars are forced to rub or slide upon the mating surface of their supporting seat bars attached to the gate frames.

The slot recesses in the gate frames are continued upward in the envelope or bonnet whose lower flanges are bolted to a flange formed on the top of the frame. The gate leaf when raised to its open position is received within this bonnet, the water passage being left free. The gate frames and bonnets are strongly ribbed both vertically and horizontally to distribute the heavy reactions set up by the hydraulic hoists in the opening and closing cycles of operation, but they are not usually designed to take the internal water pressure to which they are subjected in service. This is cared for by encircling these parts with hoops of reinforcement bars and then encasing the gates in concrete upto the joint between the top bonnet flange and the bonnet cover.

The bonnet cover is bolted to the upper flange of the bonnet and is designed to resist the maximum internal water pressure. The bonnet cover is heavily ribbed if hydraulic hoists are used to operate the slide gates. It converges into the circular flange that forms the lower head of the hydraulic hoist cylinder.

Slide gates operate against very high heads in sluices which are subject to high velocity flows. It is therefore required that the gate slots should be shaped from hydraulic considerations for minimum interference in the flow of water and the gate slots and gate lip should be shaped so that no cavitation is produced. Extensive tests[8] and practical experience have proved that the gate slots should be suitably bevelled and the gate lips should be shaped as shown in the arrangement adopted for Rihand Dam. The gate slot dimensions are further reduced by adopting the shape as shown instead of the conventoinal fish-bellied shape of slide gates.

For slide gates operating under high head, the important feature of design is provision of seal arrangement which does not bind under the high bearing loads encountered. This is achieved by using seals and seats of bronze of different composition such as US-QQ-B-691-B, composition 2 and composition 6, respectively, as developed by U.S.B.R. Even better results are achieved by using monel seals and bronze US-QQ-B-691-B composition 6 seats as used in Rihand Dam. Arrangement for lubrication of seats as used in case of Rihand Dam further increases the efficiency.

Vibrations in the gate are further reduced by cutting off all water passing over the gate during partial operation by providing a continuous plate on the inside of the downstream bonnet.

The slide gate is designed to withstand a dynamic head of 174.2ft., plus a vacuum head of 11.28 ft. Thus the maximum design head upto the centre of gate works out to 185.5 ft. of water. The structural design of slide gates is similar to other gates. Gates and gate frames made of welded construction have been used from considerations of economy.

The total weight of slide gates and frames is 51 tons. They shall be got made from foreign manufacturers according to detailed drawings and specifications issued Departmentally.

Slide Gate Hoists

One hoist for each slide gate capable of operating the gate under full unbalanced hydraulic head for which the gate is designed is provided.

With sluice gate of sliding type a considerable portion of hoisting load is gate friction. With increase in gate opening from fully closed position, downpull occurs on gate, and is maximum at 80% gate opening. At this opening the hoisting forces are maximum and consist of gate friction, downpull, weight of moving parts and hoist frictions. The capacity of hoist has been computed to be 500,000 lbs and hydraulic hoist is provided as it shall be economical and efficient.

For disturbed flow in the sluice, the sluice gate in full open position should be just flush with the sluice liner. This condition determines the lift of the gate. The minimum lifting speed is specified as 1 foot per minute.

The hoist shall be of hydraulically operated cylinder and piston type and shall use oil as the operating fluid. The hoist is located in the sluice gate operating chamber at El 722.00, just above the gate slot. The hoist is supported on bonnet cover which in turn is bolted to gate bonnet. The bonnet cover also forms the lower head of the cylinder. The hoist is provided with local and remote control indicators. The installation and the details of the hoist are illustrated in figs. 7 and 8.

For erection and repairs to the hoist an overhead travelling device shall be provided in the operating chamber. To disassemble the hoist and the gate assembly, the piston is lifted to top position and piston stem is locked by hanger. The bonnet cover bolts are disengaged and the hoist is removed.

The cylinder shall comprise of a single shell with flanges on either end. The cylinder shall be designed to operate at a pressure of 1500 psi. The hoist shall open and close the gate by applying oil from below or top of piston. The gate and moving parts of the hoist shall be supported in the open position by piston resting on oil pressure.

The design and construction of the hoist cylinder is similar to that of the intake gate hoists.

The slide gate hoist shall be operated only from the unit control cabinet located in the sluice operating chamber. Two control cabinets one for each sluice shall be provided. The control system of the hoists is similar in principle to that of the intake gate hoist except that it is very much simpler as described below.

The hoist shall be actuated by operating the four-way valve on the control cabinet. Operation of the four-way valve will also operate the pump and motor sets. The four-way valve shall have a lever which can be turned into three marked positions 'open' 'close' and 'stop'. It shall be possible to stop the gate in any position and if the gate is stopped at an intermediate position it shall be possible to actuate the hoist either for closing or opening cycle.

A limit switch shall be provided in the control system which will get a sound alarm at the Power House unit board in case the piston creeps down more than $\frac{1}{2}''$ from the top position due to leakage of hydraulic oil past piston when the gate is normally in open position. The alarm indicates that the motor and pump should be operated to bring the piston to top position.

One power cabinet is provided for the gate hoist and is installed in the operating chamber with the control cabinet. The power cabinet consists of two sets of electric motors and pumps connected in parallel, oil tank, oil filter, relief valves,check valves, pressure gauges, pressure switches, valves and fittings.

Sluice gate and hoists shall be obtained from outside India at a cost of about Rs. 1,12,000, manufactured according to detailed departmental drawings and specifications.

Sluice Fixed Wheel Gate

A fixed wheel gate is installed at the entrance to the sluice as shown in Fig. 9. The size of $4' \times 9'$ sluice at the entrance is $6'\text{-}8'' \times 14'\text{-}2\ 3/4''$ on account of intake transition and a suitable sized fixed wheel gate is provided to cover and seal this opening. Normally the fixed wheel gate shall be used to stop the flow of water into the sluice while the slide gate is under repair and to enable the repair and inspection of the slide gate, frames, guides, and conduit. For routine inspection and repair, the water pressure acting on the gate will be balanced by closing the slide gate but in emergency the gate will be closed and opened against full upstream water head.

The gate shall be operated from the top of the dam by means of the intake gantry crane. Guides and frame shall be provided for both the sluices but only one gate shall be provided.

The structural arrangement of the gate is similar to that for intake gate. Rubber seals shall be provided all round the gate on the downstream side to

bear against stainless steel seat to prevent leakage against water. Usual arrangement of gate frames is provided for guiding and seating of the fixed wheel gate. The weight of the gate is 13 tons and that of gate frames 35 tons.

MISCELLANEOUS EQUIPMENT

Draft Tube Bulkhead Gates

Bulkhead gates are provided for the draft tubes of the powerhouse to permit dewatering the draft tubes for inspection and repair of turbine and draft tube. Since all the draft tubes will not be required to be dewatered at the same time only two sets of gate shall be provided. Gate track, guides and seal seat assemblies shall however be provided in all the draft tubes.

The draft tube gates shall be operated under balanced head of water by means of a gantry crane travelling on the deck above the draft tube. When not in use they shall be kept suspended above the draft tube opening and under the draft tube gantry platform on suitable latches to be provided in unit nos. 1 and 6.

The size of the draft tube is 13.75 ft. × 20.00 ft. and the gates are made accordingly of suitable size. The structural arrangement and design of the gates is similar to that of the intake stoplogs. The gates are designed for normal tail race water head during normal operation. They shall however, also be designed for use during emergency condition with higher tail race elevation due to floods. Maximum normal water head upto bottom of gate is 30ft. and maximum emergency head is 73.45′.

Guides and seal assemblies are provided to guide the gates and to provide suitable surface against which the gates can rest to seal off the opening and thus transmit pressure to the concrete structure. Sealing is provided by bronze to bronze seal and seat and by music note rubber seal provided all around the gates. The water pressure will be acting on the downstream face of the gates forcing it against the downstream face of the draft tubes. However, on account of turbulance in the tailrace, there is sometimes tendency to counteract the sealing force. Springs are therefore, provided and located between the guide and the gates so as to provide a sealing pressure equal to 25 lbs/sq. ft. of gates.

To provide for the emergency when the gates may be subjected to water head upto reservoir level on account of running of turbines by mistake, without removing the draft tubes gates, the bolts of the guide are designed so that they get broken allowing the gates to be blown off before any damage to the gates or the concrete structure may take place.

The total weight of the draft tube gates and guides is 110 tons. They are being manufactured in the Irrigation Department Bareilly Workshop, according to detailed specifications and drawings issued to them.

Draft-tubes Gantry Crane

A 25 tons capacity, electrically operated, outdoor travelling type crane shall be provided for handling the draft-tube gates. The crane shall be located on a deck at R.L. 663.00 above the draft-tube gates. The total travel of the crane shall be 380′ and the total lift shall be about 60′. When not in use the crane shall be stored in a crane house adjoining the operation deck.

Global tenders have been invited for the supply of the crane, according to detailed design and specifications issued departmentally.

Passenger Elevator

It is proposed to install a passenger elevator in bock No. 34 for use in connection with the construction, operation, inspection and maintenance of the Rihand Dam and power plant and to carry the heavy tourist traffic. It is proposed to provide a 2500 lbs capacity passenger elevator, gearless traction type with a speed of 500 ft. per mt. The total travel of the elevator will be 258.33 ft. with seven landings.

It is also proposed to install a geared traction type passenger elevator for use in the power house with a minimum capacity of 1000 lbs. a speed of 100 ft/mt. and with a total travel of 64′.

ACKNOWLEDGEMENT

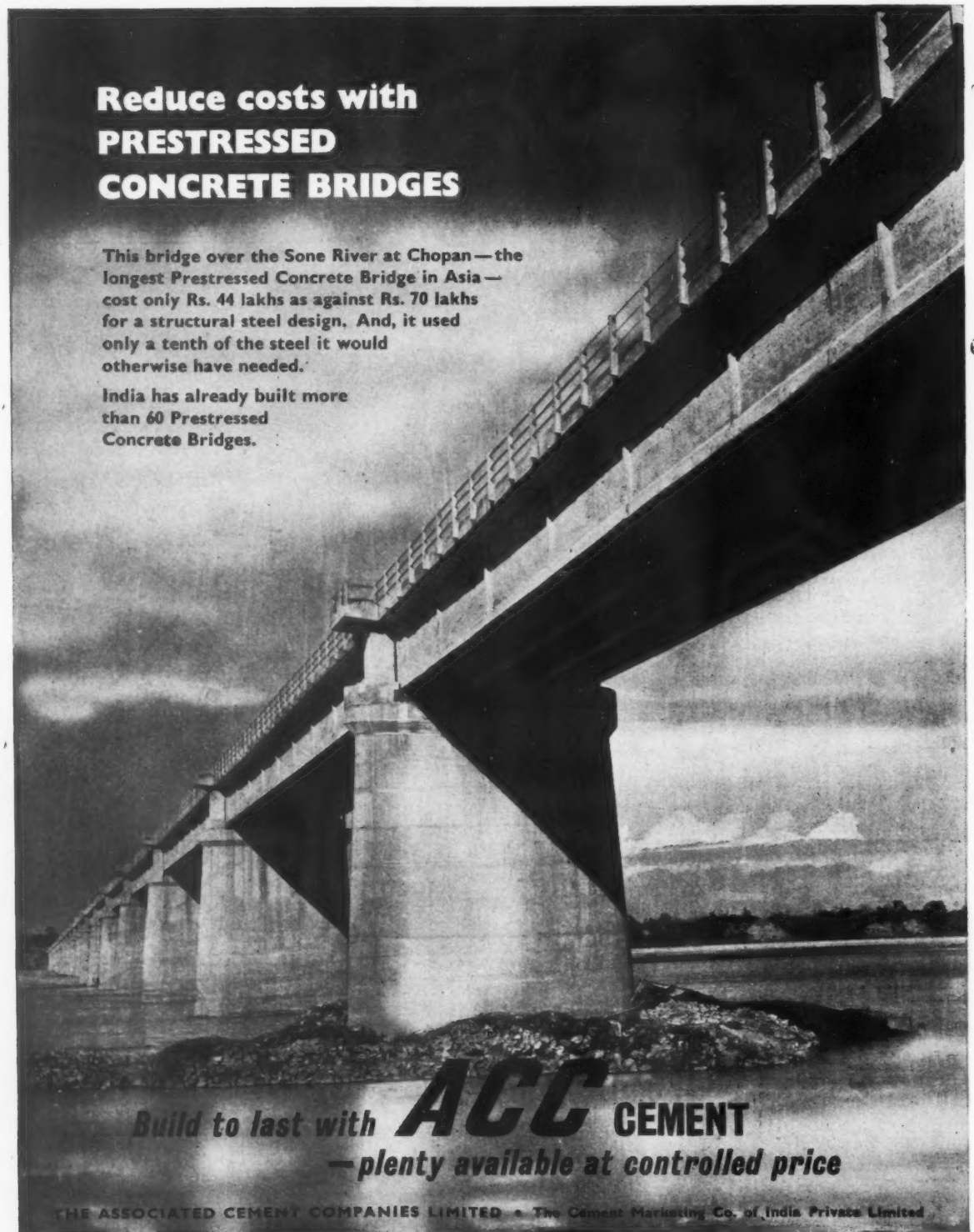
Acknowledgement is gratefully made of the assistance of Shri S. C. Sinha, Assistant Engineer particularly in getting the drawings prepared for this article.

REFERENCES

Reference may be made to the following publications issued from the Rihand Dam Design Directorate, referred to in the article also, for details of design criteria. Reference for detailed drawings may be made to specifications of the particular equipment, also issued from the Rihand Dam Design Directorate.

References

1. Design Criteria — Penstock
2. „ — Fixed Wheel Penstock Gates
3. „ — Fixed Wheel Penstock Gate Hoists
4. „ — Intake Stoplogs
5. „ — Trashrack.
6. Gates and control equipment proposed for disposal of water at Rihand Dam by M. C. Chaturvedi—Symposium held by C.B.I. in 1957.
7. Design Criteria—Spillway Tainter Gates and Hoists.
8. Design Criteria—Slide Gates and Hoists.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This bridge over the Sone River at Chopan—the longest Prestressed Concrete Bridge in Asia—cost only Rs. 44 lakhs as against Rs. 70 lakhs for a structural steel design. And, it used only a tenth of the steel it would otherwise have needed.
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Cement-concrete Mixes and their Quality Control for Rihand Dam

By K. M. MAHESHWARI

THE Rihand Dam is a gravity type cement-concrete structure 3065 feet long and 300 feet high. It is divided into 61 independent monoliths of 48 to 60 ft. in width without any longitudinal joints. The quantity of concrete involved in the dam is 2,080,000 c.yd. (or 56,000,000 cft.). For concreting structure of such a vast magnitude, the necessity of proper planning, rational design considerations, construction procedures and quality control to achieve the desired objectives at a minimum cost is apparent. In this article, various aspects of cement-concrete of which the Dam is being made, shall be presented.

OBJECTIVES

Rihand Dam, like other dams, is expected to be of service for at least 100 years. It will have to stand sustained water pressures, ravages of weather on exposed portions and the structural stresses produced due to various loadings. The cement-concrete will have to be of the required workability, consistency, durability and strength for minimum cost.

In a mass concrete structure, the temperature rise due to hydration of the cement also constitutes a serious problem, which if uncontrolled would result in objectionable thermal expansion and contraction. Our objective, therefore, is the production of impermeable and durable concrete of required strength and workability, but placed and controlled in such manner as to prevent any objectionable cracking.

DESIGN STRENGTH

What should be the design strength of concrete is an important question in the planning of any cement concrete gravity dam. The strength of lean concretes used on recent projects have exceeded the most optimistic expectations and through the use of entrained air and pozzolans, a considerable reduction in cement content has been possible.

Mass Concrete

The computed stresses in Rihand Dam for various conditions are shown in Fig. 1. In the maximum section of the dam, the maximum stresses with worst conditions come to 430 p.s.i. Allowing a safety factor of 4, the design strength of concrete comes to 1720 p.s.i. In all gravity concrete dams constructed so far, design strength of more than 2000 p.s.i. has been used. It was therefore decided to adopt 2000 p.s.i. as design strength for Rihand Dam concrete at 28 days without pozzolan and at 90 days with pozzolan.

Face Concrete

The upstream face of the dam will have to withstand the hydrostatic head of water and must be comparatively more impermeable. The upstream face concrete in 7′ thickness is therefore of rich mix of 4000 p.s.i. strength. The downstream face of the dam on the other hand will be exposed to seasonal effects of sun and air and hence the strength of concrete on this face in a thickness of seven feet has been adopted as 3,000 p.s.i.

Special Concrete

Abrasive action on concrete by high velocity waters and the danger of cavitation are expected in the spillway and sluices portion of the dam. It is an accepted fact that resistance of concrete to forces of cavitation and abrasion increases with increased compressive strength of the concrete. An average strength of 6000 p.s.i. in top one foot thickness of concrete is provided in the following locations, where pitting due to cavitation is likely to occur and the velocities would be more than 40 feet per second.

(i) Spillway crest above EL. 800.

(ii) Spillway bucket below EL. 700.

(iii) Sluices in one foot thickness all around inner surfaces.

This extra rich concrete corresponds to the recommendations of A.C.I. Committee 210 (Erosion resistance of concrete in hydraulic structures, title No. 52-18).

Design strengths of reinforced concrete have been fixed on the basis of working stress as 750 p.s.i., and safety factor of 3. Also the average strength of such concrete was required to have a factor of safety of 4 and therefore the average strength works out to be 3000 p.s.i.

Various types of concrete mixes, depending on the maximum size of aggregates, placeable in a particular location and the desired strength being used at Rihand Dam are tabulated in table 1.

TABLE I

TYPES OF CONCRETES PROPOSED FOR VARIOUS LOCATIONS IN RIHAND DAM

Note:—'A' denotes aggregates, suffix of 'A' denotes maximum size of Aggregates, 'S' denotes strength and suffix of 'S' denotes average strength of concrete at 28-days in hundreds of lbs. per sq. inch.

S.No.	Locations.	Type of *concrete*
1.	Mass concrete in Dam and spillway except 7′ on all faces and 3′ around the galleries, penstocks, shafts, river outlets and other openings.	A_6S_{25}
2.	Three feet all around foundation galleries.	A_3S_{30}
3.	Three to five feet thick around penstocks, three feet around galleries other than foundation galleries, shafts and other openings in the dam which have reinforcement around them.	$A_3/_2S_{30}$
4.	Seven feet thick on U.S. face.	A_6S_{10}
5.	Seven feet thick on D.S. face excluding crest and D.S. face of spillway.	A_6S_{30}
6.	Seven feet thick on D.S. face of spillway between elevations 800 and 700.	A_3S_{40}
7.	Spillway crest above EL. 800 (a) One foot thick on surface.	$A_3/_4S_{60}$
	(b) Four feet below top one foot.	A_3S_{40}
8.	Spillway bucket below EL. 700.	
	(a) One foot thick below top surface.	$A_3/_4S_{60}$
	(b) Four ft. thick under one ft. from top surface.	A_3S_{40}
	(c) All concrete under five ft. from top surface.	A_6S_{25}
9.	Sluice outlets.	
	(a) One ft. thick all around inner surface.	$A_3/_4S_{60}$
	(b) Two ft. thick all around and under 1 ft. thick inner surface.	$A_3/_2S_{60}$
10.	Training walls of spillway on Dam.	A_3S_{40}
11.	Training walls.	
	(a) Un-reinforced D.S. of spillway.	A_6S_{30}
	(b) Reinforced.	A_3S_{30}
12.	Concrete in piers.	A_3S_{40}
13.	Trash-rack structure.	$A_3/_2S_{30}$
14.	Side walls, curb, parapets.	$A_3/_2S_{30}$
15.	Block-outs.	$A_3/_4S_{40}$
16.	Formed Holes 6″ or more dia.	
	(a) Other than drainage holes.	A_3S_{30}
	(b) Drainage holes.	A_6S_{25}
17.	Concrete in bridge over spillway.	$A_3/_2S_{30}$
18.	Retaining walls for parking area and tail race.	
	(a) Un-reinforced.	A_6S_{25}
	(b) Reinforced.	$A_3/_2S_{30}$
19.	Power House sub-structures.	
	(a) Un-reinforced.	A_3S_{25}
	(b) Reinforced.	$A_3/_2S_{30}$
20.	Power House Super-structures.	
	(a) Thickness more than 6″.	$A_3/_2S_{30}$
	(b) Thickness less than 6″.	$A_3/_4S_{30}$
21.	Concrete in switchyard for foundation of structures.	$A_3/_2S_{30}$
22.	Concrete in cable tunnel.	$A_3/_2S_{30}$
23.	Concrete reinforced in transformer deck.	
	(a) More than 6″.	$A_3/_2S_{30}$
	(b) Less than 6″.	$A_3/_4S_{30}$

Strength Tests and Test Moulds

Since Rihand Dam has been divided into independent monoliths, which have height more than twice the average width of the section and less than twice the bottom base width, the test specimen for predicting the field strength of concrete should be such that its height is twice its base width. For convenience in making moulds for test specimens and to have uniformity with core-drills of field concrete it was decided to have cylinder moulds. The common moulds adopted have been 6″ dia × 12″ high while bigger moulds 18″ dia × 36″ high have also to be prepared for determination of actual strength of concrete containing 6″ maximum size of aggregates. The type of concrete to be filled in the moulds has a maximum size of aggregate equal to $\frac{1}{4}$th the diameter of the mould. In case of concretes containing 6″ and 3″ maximum size aggregates, screening on $1\frac{1}{2}$″ sieve is done and moulds cast from minus $1\frac{1}{2}$″ concrete. Since the water-cement ratio in original and wet screened concretes is not altered, the same strength can be expected in both cases. Recently some 6″ dia. cores were drilled from hardened concrete in the Dam at 28 days and tested. The strength of concrete from the Dam was found to be 5% higher than the similar test specimens, made from the same concrete but cured in laboratory fog-room. Cube specimens have not been used in Rihand Dam for reasons mentioned above although these give a strength about 20% higher than the cylinder strength.

Rational Basis of Obtaining Desired Strength

The strengths of cement-concrete mixes as proposed have to be ensured in field, where variations in quality of ingredients, moisture in aggregates, weighing of ingredients, unit weight of materials etc., introduced some variations in the strength of concrete. Let us take the typical example of change in moisture in aggregates. The water-cement ratio and therefore the strength of concrete changes being higher or lower than the design strength due to variation in total water content in the mix because of variation in the aggregate moistures. It is therefore reasonable to fix a limit for the lowest strength of

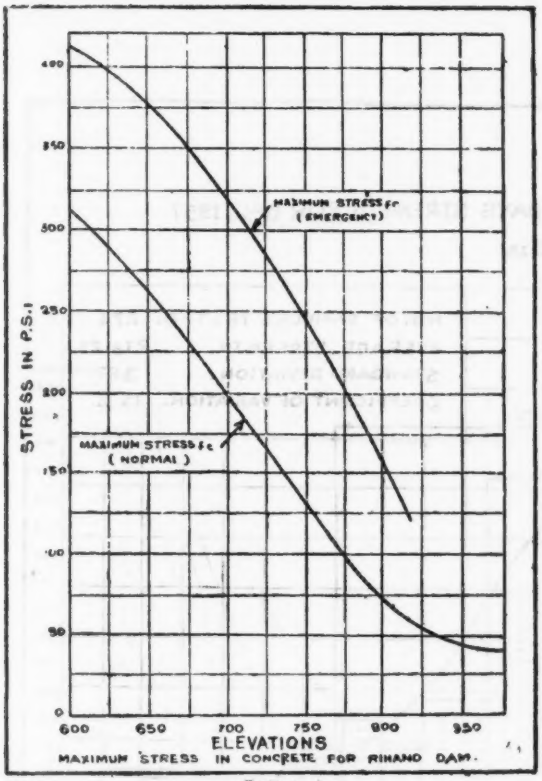

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FIG. 1

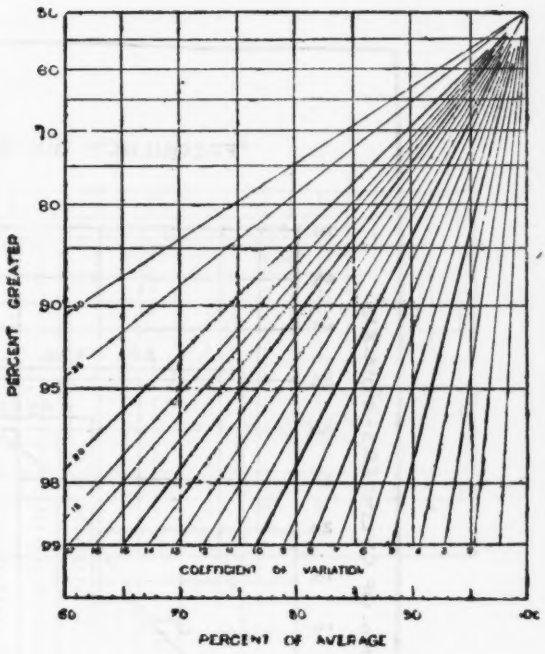


FIG. 3

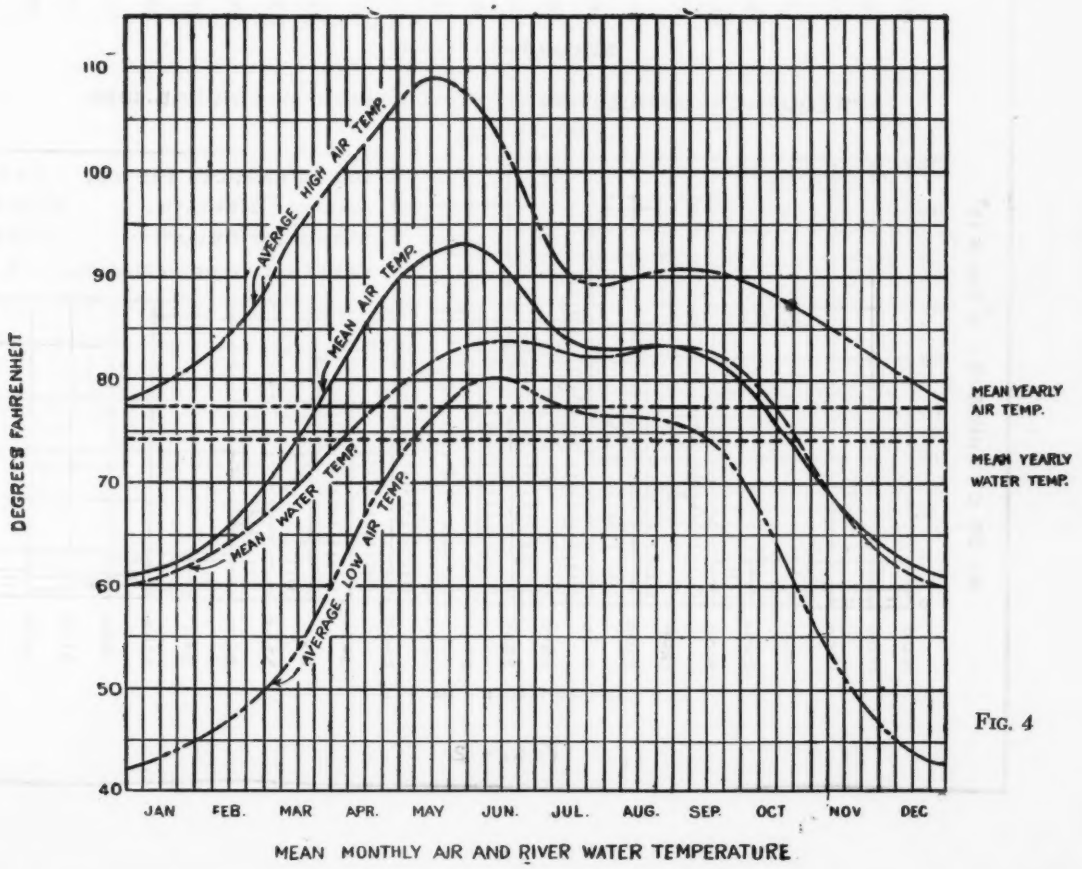
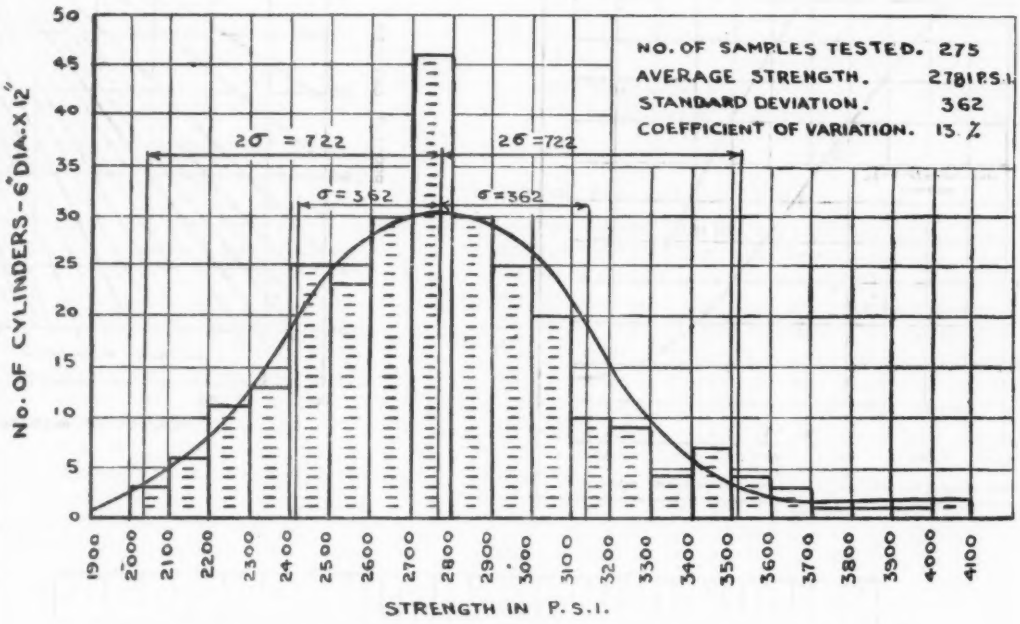

DEGREES FAHRENHEIT
AVERAGE HIGH AIR TEMP.
MEAN AIR TEMP.
MEAN WATER TEMP.
AVERAGE LOW AIR TEMP.
MEAN YEARLY AIR TEMP.
MEAN YEARLY WATER TEMP.
JAN FEB. MAR APR. MAY JUN. JUL. AUG. SEP. OCT NOV DEC
MEAN MONTHLY AIR AND RIVER WATER TEMPERATURE

FIG. 4

FREQUENCY DISTRIBUTION 28 DAYS STRENGTH FOR DEC. 1957

MIX $A_6 S_{25}$


NO. OF SAMPLES TESTED. 275
AVERAGE STRENGTH. 2781 P.S.I.
STANDARD DEVIATION. 362
COEFFICIENT OF VARIATION. 13 %
2σ = 722
2σ = 722
σ = 362
σ = 362
No. OF CYLINDERS - 6" DIA. X 12"
STRENGTH IN P.S.I.

FREQUENCY DISTRIBUTION 28 DAYS STRENGTH FOR FAB. 1958

MIX. $A_6 S_{25}$

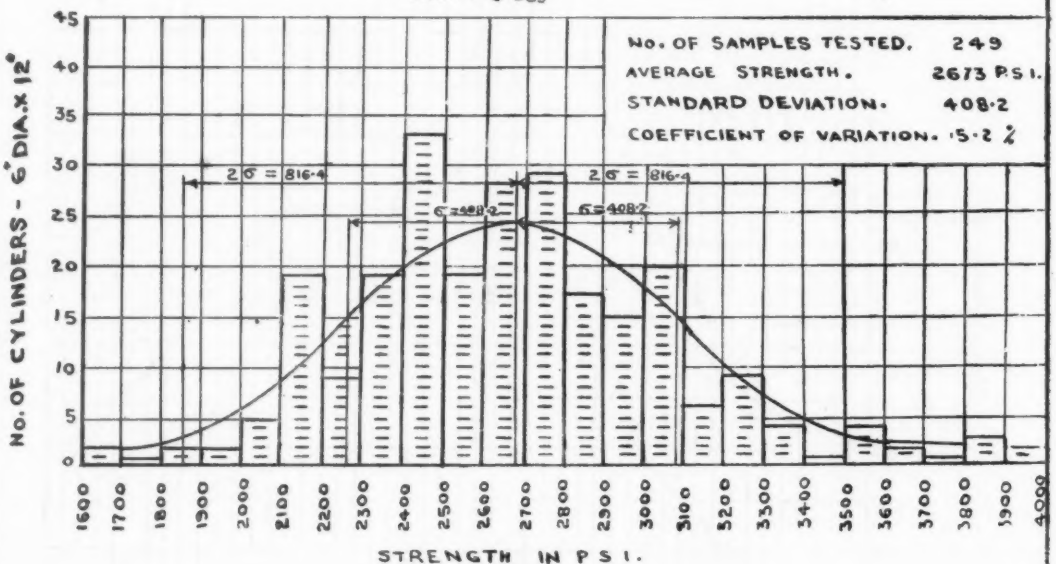
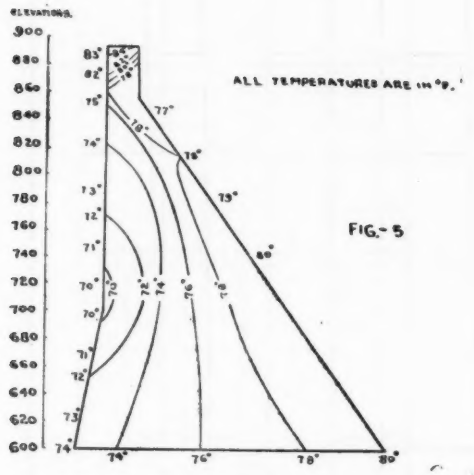
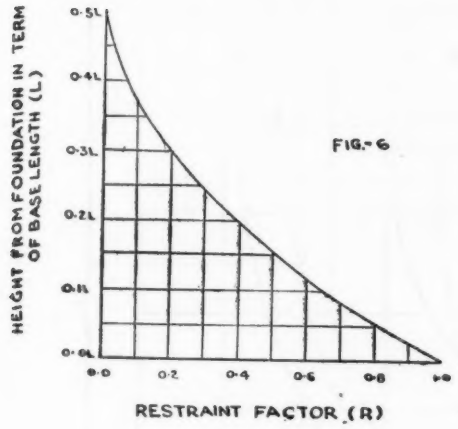

NO. OF SAMPLES TESTED. 249
AVERAGE STRENGTH. 2673 P.S.I.
STANDARD DEVIATION. 408·2
COEFFICIENT OF VARIATION. 15·2 %
2σ = 816·4
2σ = 816·4
σ = 408·2
σ = 408·2
No. OF CYLINDERS - 6" DIA. X 12"
STRENGTH IN P.S.I.

FIG. - 2



COMPUTED STABLE TEMPERATURE IN RIHAND DAM



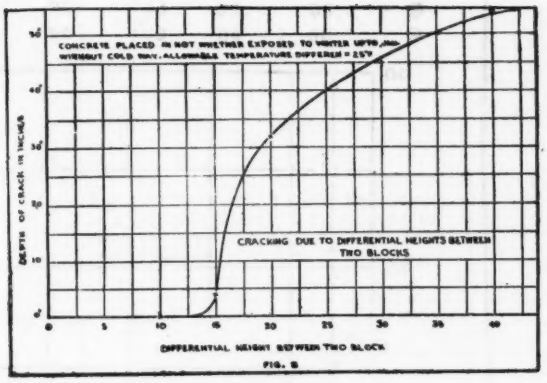

CONCRETE PLACED IN HOT WHETHER EXPOSED TO WINTER UPTO
WITHOUT COLD WAY. ALLOWABLE TEMPERATURE DIFFERENCE = 25°F
DEPTH OF CRACK IN INCHES
CRACKING DUE TO DIFFERENTIAL HEIGHTS BETWEEN TWO BLOCKS
DIFFERENTIAL HEIGHT BETWEEN TWO BLOCK
FIG. 8

FIG. 8

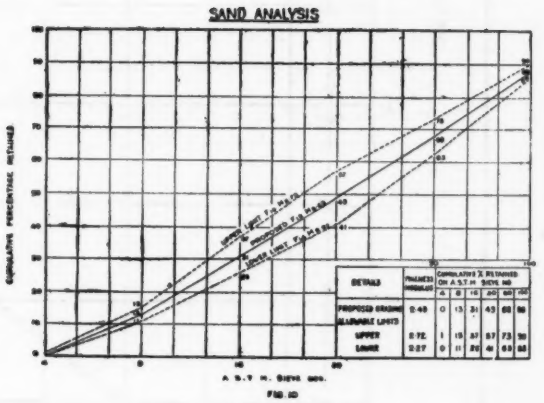


FIG. 10

TEMPERATURE DISTRIBUTION UNDER CERTAIN CONDITIONS.

	PLACEMENT TEMP.	AMBIENT			LIFTS AT 3 DAYS INTERVAL
①	60°F	60°F	MID WINTER WITH COOLING		
②	-DO-	70°F	-DO-	WITHOUT -DO	3 -DO-
③	-DO-	60°F	-DO-	71·5°F	SPRING & AUTUMN WITH COOLING 4 -DO-
④	-DO-	60°F	-DO	84°F	HOTTEST SUMMER -DO- 5 -DO
⑤	-DO-	60°F	-DO-	84°F	-DO- -DO- -DO- 4 -DO-
⑥	-DO-	60°F	-DO	84°F	-DO- -DO- -DO- 3 -DO-

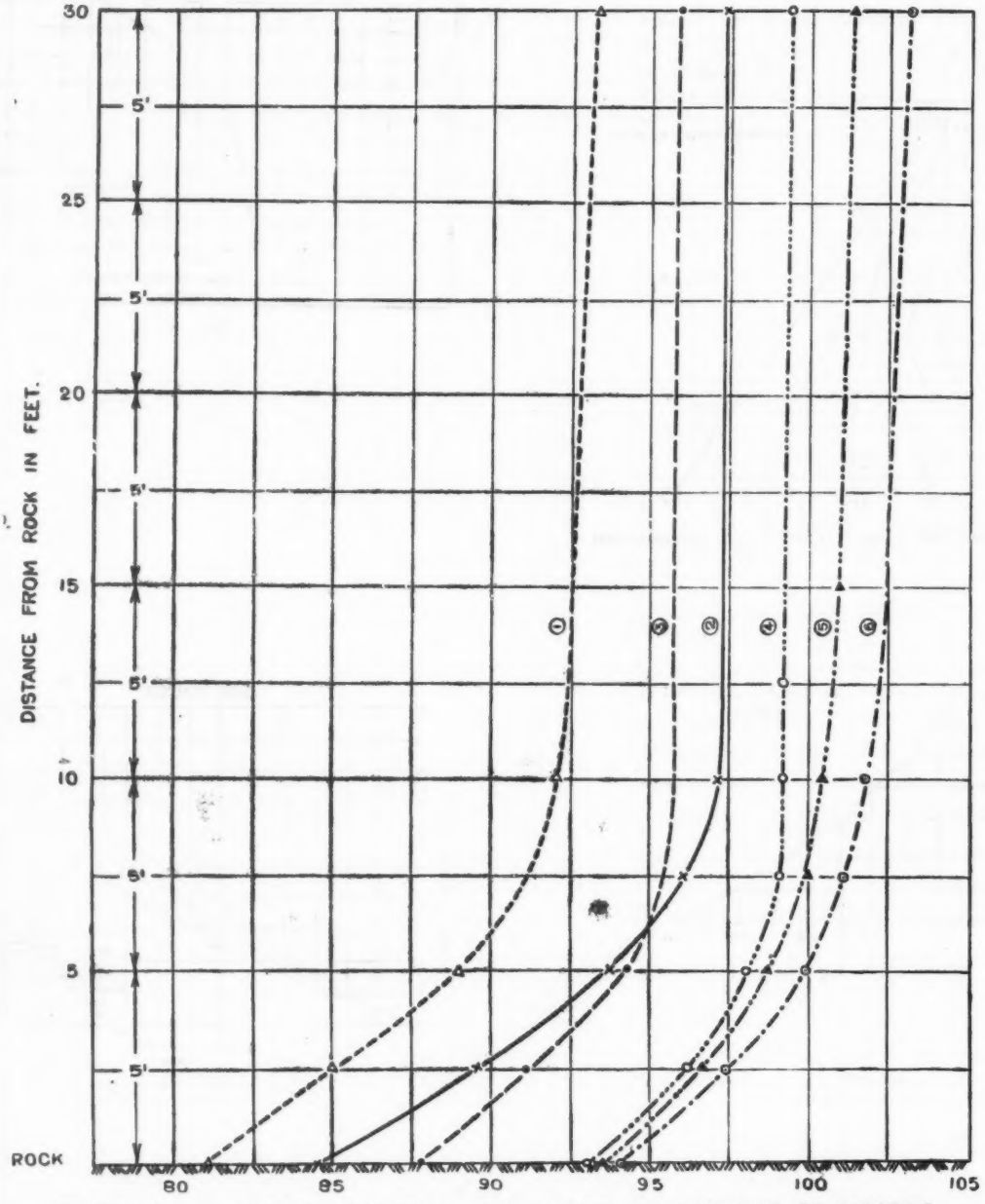
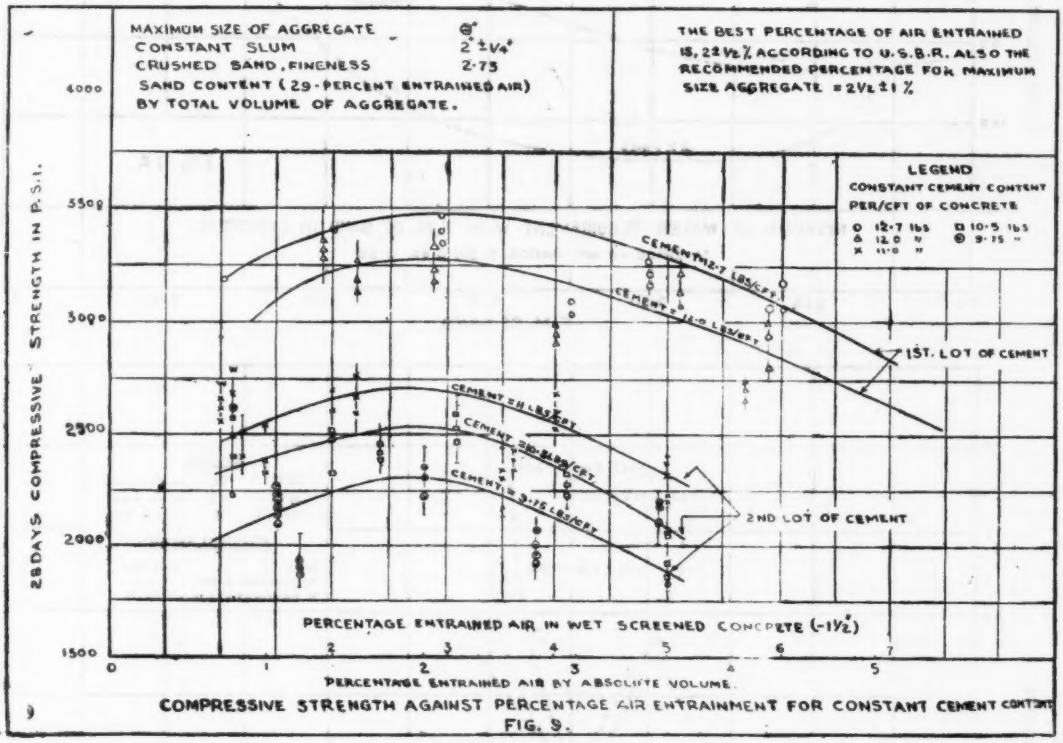


FIG. 7


MAXIMUM SIZE OF AGGREGATE 6"
CONSTANT SLUM 2" ±¼"
CRUSHED SAND, FINENESS 2·73
SAND CONTENT (2% PERCENT ENTRAINED AIR) BY TOTAL VOLUME OF AGGREGATE.
THE BEST PERCENTAGE OF AIR ENTRAINED IS, 2½% ACCORDING TO U.S.B.R. ALSO THE RECOMMENDED PERCENTAGE FOR MAXIMUM SIZE AGGREGATE = 2½ ± 1 %
LEGEND
CONSTANT CEMENT CONTENT PER/CFT OF CONCRETE
12·7 lbs
12·0 "
11·0 "
10·5 lbs
9·75 "
28 DAYS COMPRESSIVE STRENGTH IN P.S.I.
4000
3500
3000
2500
2000
1500
CEMENT=12·7 LBS/CFT
CEMENT=12·0 LBS/CFT
CEMENT=11·0 LBS/CFT
CEMENT=10·5 LBS/CFT
CEMENT=9·75 LBS/CFT
1ST. LOT OF CEMENT
2ND LOT OF CEMENT
PERCENTAGE ENTRAINED AIR IN WET SCREENED CONCRETE (-1½")
0 1 2 3 4 5 6 7
PERCENTAGE ENTRAINED AIR BY ABSOLUTE VOLUME.
0 1 2 3 4 5

COMPRESSIVE STRENGTH AGAINST PERCENTAGE AIR ENTRAINMENT FOR CONSTANT CEMENT CONTENT

FIG. 9.

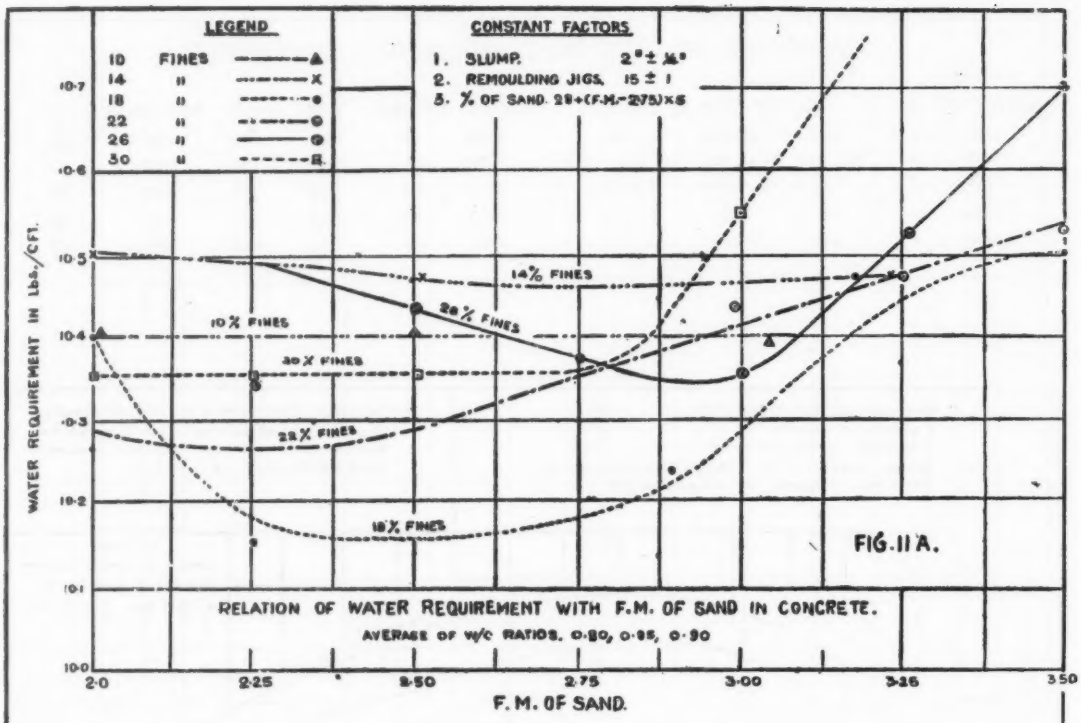


FIG. II A.

RELATION OF WATER REQUIREMENT WITH F.M. OF SAND IN CONCRETE.

AVERAGE OF w/c RATIOS. 0·80, 0·85, 0·90

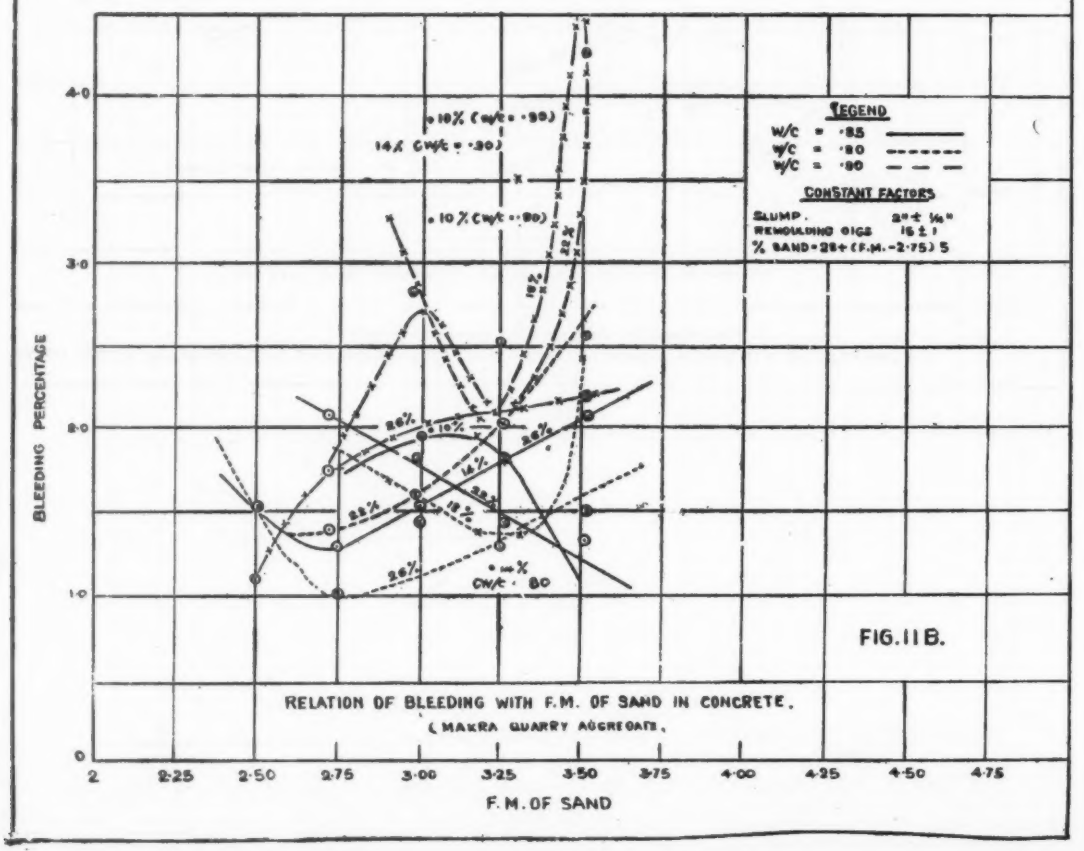
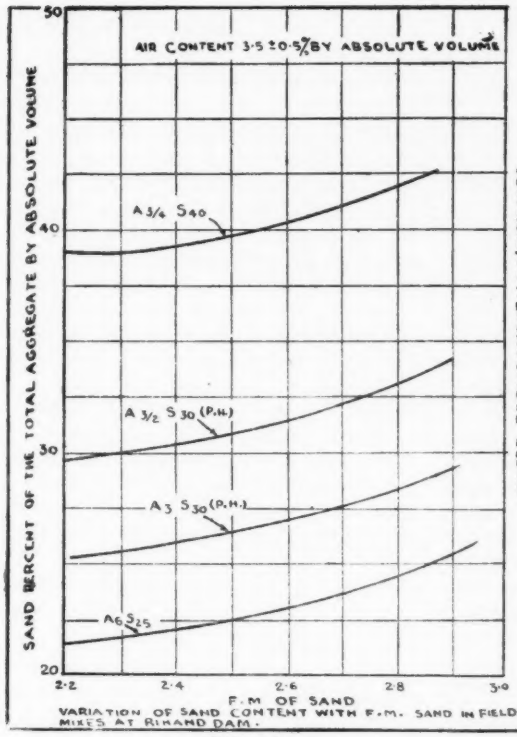


FIG. II B.

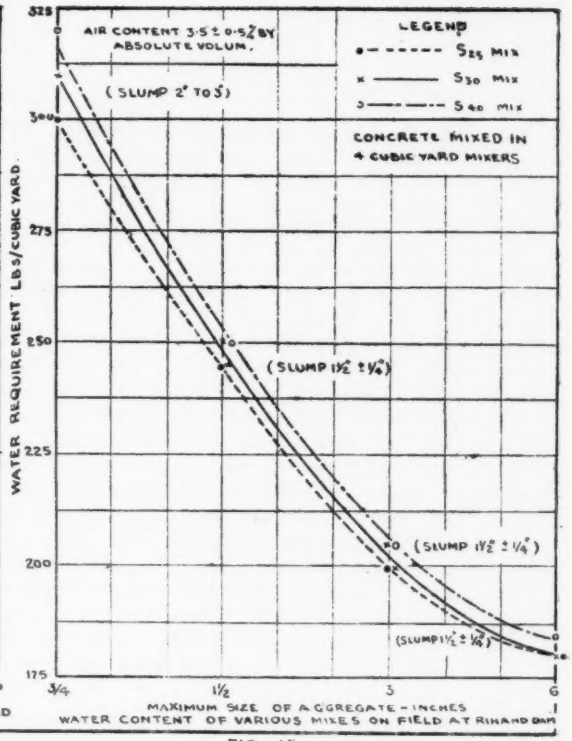
RELATION OF BLEEDING WITH F.M. OF SAND IN CONCRETE.

(MAKRA QUARRY AGGREGATE.



VARIATION OF SAND CONTENT WITH F.M. SAND IN FIELD MIXES AT RIHAND DAM.

FIG. 12



WATER CONTENT OF VARIOUS MIXES ON FIELD AT RIHAND DAM

FIG. 13

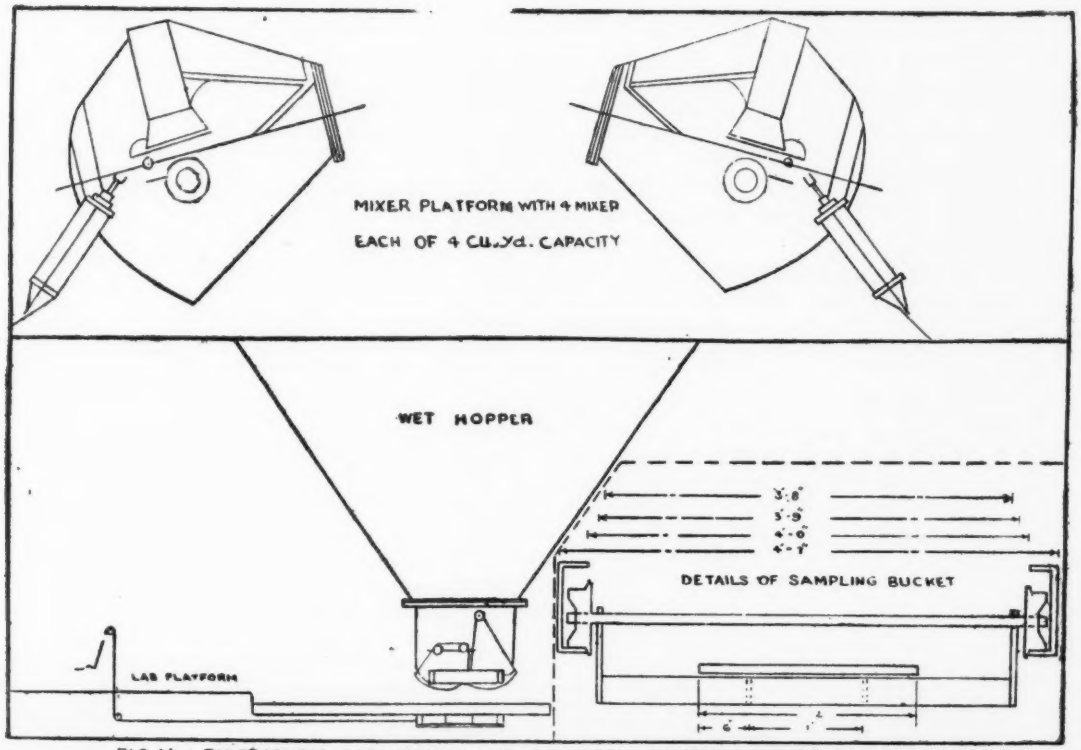


FIG.14 - DIAGRAMATIC ARRANGEMENT OF CONCRETE SAMPLING BUCKET AT BATCHING PLANT.

concrete and its frequency corresponding to a certain design strength.

Various practices have been in vogue as regards the strength of concrete and its variation in field. The conservative view is to be oversafe and design the concrete on a certain minimum strength basis, which is generally the average strength or the design strength. The United States Bureau of Reclamation (U.S.B.R.) follow a practice according to which 80 percent of test specimens may be above and 20 per cent below the designed strength but no limit on the minimum strength was been specified by them. The minimum strength for Rihand Dam concrete has been adopted as per Indian Standard specification which states that the failure stress of concrete is equal to the lowest strength which must be exceeded by 99 per cent of the specimens = 75% of design strength. The design criteria for variation in strength of Rihand Dam concrete are therefore as follows:—

Strength of 90 percent specimens to be above design value.

Strength of not more than 10 per-cent specimens to be below design value.

Strength of not more than 1 per cent specimens to be below the minimum value.

The strength of test moulds varies above and below the desired strength and falls in some pattern of a normal probability curve as shown in fig. 2. With good control the strength values are bunched close to the average and the curve is steep. With poor control the values are spread out laterally and the curve is flattened. The radius of gyration of the points about the centre is called the 'Standard deviation' which divided by the average value gives the 'coefficient of variation'.

Let the number of specimens be 'n' which should be at least 30.

$$X = \text{average strength of concrete} = \frac{\Sigma_o^n X_n}{n}$$

$$\text{Standard deviation } \sigma = \sqrt{\frac{\Sigma_o^n (X_n - X)^2}{n}}$$

and the coefficient of variation in percentage

$$V = \frac{\sigma}{X} \times 100$$

For good control the coefficient of variation in the field has been found to be from 10 to 15 per cent in U.S.A. and for safety the figure of 15 be adopted for Rihand Dam.

An example of computing strengths of concrete mix from probability variation curve (Fig. 3) is given now. For mass concrete, the design strength has been adopted as 2,000 p.s.i. For 90 per cent specimens to have strength above the design strength and the coefficient of variation as 15 per cent, the design strength would be 80 per cent of the average strength (Fig. 3). Thus the average strength of mass concrete should be $\frac{2,000}{0.80}$ = 2,500 p.s.i. According to Indian Standard specifications the average strength should be 1.25 times the required field strength which also comes out to be 2,500 p.s.i.

TEMPERATURE CONTROL

Cement-concrete develops strength by the hydration of cement which continues over a long period after the cement and water come into contact. During the process of hydration of cement heat is generated which in thin structures is dissipated easily through exposure and curing but takes a long time for the excess heat to be lost in massive concrete structures. The temperature of massive concrete structure like a gravity dam continues to rise over a long period and then gradually cools down to a stable temperature. During the early age, the creep is high and modulus of elasticity low with the result that the compressive stresses induced in concrete due to expansion are within safe limits, but at later age the contraction in concrete induces tensile stresses which, since the creep is low and modulus of elasticity high, would result in objectionable cracking, if uncontrolled.

The time and rate of temperature rise or fall depends upon the size of the structure, the quantity and type of cement used and on the methods employed for the dissipation of heat.

The chemical analysis and actual heat of hydration values of Churk cement used in Rihand Dam concrete have been adopted to estimate the heat of hydration in the various types of concretes. Since the mass concrete in the dam occupies more than 80 percent of the total volume, the temperature control has been considered on the basis of heat evolution and dissipation in mass concrete filling whole of the dam.

Air and water temperature of Rihand Dam site are shown in fig. 4.

The expected stable temperatures as computed on considerations of air, water and seasonal temperatures are shown in fig. 5.

Foundation Restraint

The foundation of each block is bounded to the rock while the top is free. In other words we can say that the foundation of each block is restrained such that the volume change of concrete at this point cannot occur and stresses are created due to 100% rigidity at the foundation. The restraint factor varies as shown in fig. 6 from unity at the base to zero at a height equal to half the base length.

Maximum Temperature Rise

Thus the importance of difference in maximum temperature rise and the final stable temperature is apparant. The tensile stresses induced in restrained concrete are given by the formula:

$$F = (t_p + t_r - t_f)\, E_s .C.R. \quad \text{.....(i)}$$

where f = tensile stress in p.s.i.
t_p = placing temperature of concrete.
t_r = temperature rise in concrete.
t_f = stable temperature in dam
E_s = sustained modulus of elasticity.
$= 2 \times 10^{-6}$ p.s.i.
C = Coefficient of linear expansion
$= 5 \times 10^{-6}$
R = Restraint factor.

The Rihand Dam mass-concrete can take 250 p.s.i. tension after allowing for other factors.

Therefore $250 = (t_p + t_r - t_f) \times 2 \times 10^6 \times 5 \times 10^{-6} \times R$

or $t_p + t_r - t_f = \dfrac{25°F}{R}$ (ii)

At the foundation $(t_p + t_r - t_f) = 25°F$ or the maximum temperature in concrete at the foundation should not rise to more than 25°F above the final stable temperature. Temperatures beyond the limits may induce cracking in the dam and endanger its safety.

Methods of Temperature Control

The foremost method adopted at Rihand Dam is to precool the concrete before placing to limit the temperature rise. No post-cooling of concrete is done. Computations indicate that placement temperature of 60°F of concrete in hottest weather can be acceptable if a suitable placement schedule is followed (fig. 7). Therefore all the ingredients of concrete except cement are cooled to such extent that the temperature of concrete at site is 60°F. Typical computations as per prerequisites of properly cooled mass concrete are shown in table 2.

Table II

COMPUTATIONS OF TEMPERATURE OF FINISHED CONCRETE A_6S_{25} MIX CONTAINING 272 LBS. CEMENT PER CU.Y.D.

Table 2 (A)

Sl. No.	Ingredients.	Weight of ingredients lbs/cu.y.d.	Moisture in ingredients, % by wt.	Weight of water due to moisture lbs.	Temperature °F	Temperature with reference to 60°F as datum °F.
1.	6″-3″ aggregate	990	0.2	2	50	-10
2.	3″-1½″ ,,	610	0.4	2	50	-10
3.	1½″-¾″ ,,	610	0.8	5	50	-10
4.	¾″-⅜″ ,,	420	1.5	6	50	-10
5.	⅜″-3/16″ ,,	305	4.0	12	64	+ 4
6.	Sand.	840	9.0	75	67	+ 7
7.	Maxing water	180 minus moisture in aggregates.	—	78	37	-23
8.	Cement.	272	—	—	95	+35

Table 2 (B)

Sl. No.	Ingredients.	Unit weight. ibs/cft.	Weight per cu. yd., lbs.	Specific heat.	Heat capacity B.Th.U.	Temperature difference from datum °F	Total heat above datum B. Th. U.
1.	6″-⅜″ aggregates	175	2,630	0.17	447.1	-10	-4,471
2.	⅜″-3/16 aggregates	175	350	0.17	51.8	+ 4	+ 207
3.	Sand.	167	840	0.17	142.8	+ 7	+1,000
4.	Cement	197	272	0.11	29.9	+35	+1,050
5.	*Moisture.*						
	(a) on 6″-⅜″ aggregates	62.5	15	1. 0		-10	- 150
	(b) on ⅜″-3/16″ fraction.	Do	12	1. 0	180.0	+ 4	+ 48
	(c) on sand.	Do	75	1. 4		+ 7	+ 525
6.	Mixing water	Do	78	1. 0		-23	-1,794
7.	Total. (unit weight of concrete)	156.6	4,227 (average for concrete.)	0.20	825.6	-4.2	-3,585

8. Heat of hydration in time taken upto Placement. $272 \times 5 = +1,360$
9. Mechanical energy converted into heat in 50 H.P. mixer mix with mixing time 3 minutes
$\dfrac{42.4 \times 50 \times 3}{4}$
$= +1,590$ Total $= +2,950$ B.T.U.
$= 3.5°F$
10. Hence temperature of concrete at site $= 60 - 4.2 + 3.5 = 59.5°F$

The first four fractions of aggregates are cooled by spraying water at 36 to 38°F while 3/8″-3/16″ fraction and sand are cooled by refrigerated air. The mixing water used in concrete is refrigerated and brought to about 37°F. Slight changes in cooling schedule are made for cooling mixes other than mass concrete.

Concrete placement programme has been prepared in accordance with the temperature control conditions. Generally concrete is placed in lifts of 5 feet thickness and curing by water-spray is done till the next lift is poured. Computations have been done to estimate the maximum temperature of concrete under different placement conditions. The methods employed in these computations are:—

(i) Definite integral method.
(ii) Carlson's method.
(iii) Schmidt's method.

The three methods give nearly identical results but for ease in computations, Schmidt's method was

employed in most of the cases. This formula involves simple constants and can be written as:—

$$t = \frac{x^2}{2h^2} \qquad \text{(iii)}$$

where t = time interval,
x = the length element or space interval,
h^2 = diffusivity constant
$= \frac{k}{sp}$ where k = conductivity
s = specific heat
and p = specific gravity

The rate of temperature change of any interior element depends only on its own temperature and on that of its two adjoining elements. With the choice of space and time interval chosen accordingly to Schmidt's formula, the temperature relationship within each group of the three elements is such that the temperature of the middle element at the end of time interval is approximately the average of the temperatures of the elements on either side at the beginning of the time interval.

In fig. 7 are shown the temperature for certain conditions of placement, interval, curing etc. The stable temperature in the Dam at the foundation in the maximum section may be 74°F and allowing for maximum rise of temperature of 25°F, the maximum temperature at the foundation should not go above 90°F. Applying creep theory to these temperature stresses, it has been proved that the maximum temperature in concrete need not be considered but instead 17 days temperature of concrete may be considered as $t_p + t_r$ to safeguard against cracks.

By computations, it has been found that the placement interval of 5 days in hot weather is just safe for the foundation concrete. This period may be reduced to 4 days in higher lifts and to 3 days in much higher lifts where retraint factor is much less. No precooling may be necessary in certain parts of cold weather and 3 days placement interval is safe in the winter season.

The restraint in concrete becomes almost unity at any elevation if it is left exposed to weather for a long time. In winters, further temperature control precautions are necessary if a lift is left uncovered for longer periods as the lift loses much of the excess heat and appreciable temperature gradients are likely to develop. Foundation rock is also cooled to the expected stable temperature before placement of Concrete so that the joints in the rock are not affected in the long life of the dam, and thus do not induce stresses in the foundation concrete. The concrete in humps and difficult locations in foundations is also poured without laying full lift. This practice allows the concrete to lose its excess heat easily and it is cured such that the temperature of concrete is near about 80°F so that the shrinkage of concrete ultimately is very small. This ensures good contact of concrete to the irregular foundation rock.

In order to safeguard against cracks in cold weather due to sharp fall in atmospheric temperature for a long time the lifts are placed at 3 days interval. The top of the lift is thus not allowed to cool below the safe limit and prevent an excessive temperature gradient in the lift itself as the interior of the lift and the block is still hot. In such weather, of course surface cracks in the sides of the block exposed to air may occur due to thermal shock but their depth will not be appreciable. Even then it is desirable that the cracking may be minimum and may not develop further due to some unforeseen or uncontrolled factors. The differential height between two adjacent block for such considerations has been limited to 20 feet (Fig. 8.).

The use of low cement content and pozzolanic materials also reduces the heat of hydration. Every effort is being made to have lowest cement content by proper control of concrete mixes while the pozzolan is in use from December 1958.

CEMENT

Cement for Rihand Dam is supplied from Government cement factory, Churk which is 50 miles from damsite. The cement conforms to the Indian Standard Specifications for ordinary Portland cement and its average properties are given below:—

(i)	Chemical Analysis:—	%
	Loss on ignition	2.6
	SiO_2	22.6
	Fe_2O_3	2.8
	Al_2O_3	6.4
	CaO	61.3
	SiO_3	2.4
	Free CaO	1.0
(ii)	Theoretical compounds:	%
	Tricalcium Silicate (Ca_3Si)	20.0
	Dicalcium Silicate (Ca_2Si)	50.0
	Tricalcium aluminate (Ca_3Al)	12.0
	Tetra-calcium-alumino-ferrite (Ca_4AlFe)	8.5
(iii)	$\frac{CaO}{2.8\ SiO_2 + 1.2\ Al_2O_3 + 0.66\ Fe_2O_3}$	= 0.85
	$\frac{Al_2O_3}{Fe_2O_3}$	= 2.28
(iv)	Initial set	3 hrs.
	Final set.	4 hrs.
(v)	Fineness	3,000 sq.cm.per.gm.
	Residue on B.S.S.	170 sieve 7%
(vi)	Specific gravity	3.12 to 3.15.
(vii)	Heat of Hydration by heat of solution method:—	
	3 days.	52 cal. per gm.
	7 days.	61 cal. per gm.
	28 days.	80 cal. per gm.

We have found Bogue's theoretical compound

computations and their effect on strength developing characteristics of cement very useful in predicting strength-age relationship. It may be interesting to discuss this basis in brief at this stage.

When cement raw materials containing proper proportions of the essential oxides are ground to a suitable fineness and then burned to incipient fusion in a kiln, chemical combination takes place, largely in the solid state, resulting in a product named clinker, which when ground with gypsum 3 to 4% to a suitable fineness is called Portland Cement. The constitution of Portland Cement clinker i.e. the compounds formed at incipient fusion have been a controversial subject and there are various theories on this subject. The most accepted view is that the essential compounds are silicates of lime: tricalcium silicate (3 Ca0. $Si0_2 = Ca_3Si$), dicalcium silicate (2 Ca0. $Si0_2 = Ca_2Si$). Also present are tri-calcium aluminate (3 Ca0. Al_2O_3; $Fe_2O_3 = Ca_3Al$) and tetra-calciumalumino ferrite (4 $Ca0.Al_2O_3$. $Fe_2O_3 = Ca_4AlFe$.) It has been demonstrated that Ca_3Si and Ca_2Si control most of the strength developing characteristics of a cement. Also Ca_3Si contributes to early strength while Ca_2Si imparts strength at later ages. The higher the sum of Ca_3Si plus Ca_2Sl, the higher is the ultimate strength of concrete, while Ca_3Al contributes practically to no strength and most of the heat of hydration.

Cement Testing and Control

As the quality of cement from the factory varies to some extent, it is necessary to keep control of the quality in order to economise on the cement content in the concrete mixes. Representative samples of cement are collected daily from the bulk cement-carriers, bringing cement from the factory to the damsite and tested at the Rihand Research Laboratory according to the Indian Standard Specification. The compressive strength of mortar and tensile strength of briquettes are tested at 3 and 7 days. Complete chemical analysis of cement is done twice in a week and the theoretical compounds in cement determined. Percentages of Ca_3Si and Ca_2Si are watched and their sum in Churk cement is about 70%. Adjustments in the cement content in concrete mixes are made if variations in the quality of cement are indicated by these tests.

Water-Cement Ratio

According to Abram's law, the strength of concrete depends almost entirely on the water-cement ratio. Lower the water-cement ratio of a cement paste, higher will be its strength. Laboratory studies of a particular cement are necessary to determine the water-cement $\frac{W}{C}$ ratio and strength curve. Probable minimum values of $\frac{W}{C}$ ratio and strengths for Churk cement are given below:—

Table 3

Water-cement ratio by wt.	Minimum Compressive Strength of 6″ × 12″ cylinders at 28 days p.s.i.	
	Air entrained concrete.	Non-air entrained concrete.
0.40	4300	5000
0.45	3850	4500
0.50	3400	4000
0.55	3000	3600
0.60	2650	3250
0.65	2300	2800
0.70	2000	2400

Cement Content

The mass concrete at Rihand Dam has 6″ maximum size aggregate. The water requirement using 4.5 – 5% entrained air in the minus 1½″ portion of the concrete mix prepared with crushed coarse aggregates and blended river and crushed sand is 180 lbs. per .cu. yd. Therefore for an average strength of 2500 p.s.i. the cement content (at 0.63 water cement ratio) used is 286 lbs. per cu. yd. of concrete, while cement content upto 265 lbs. per cu.yd. has been used when better quality of cement was available. Face concretes have higher cement contents for greater watertightness and durability according to the design criteria described before. A reduction of about 20% by weight in the cement content was possible by use of pozzolans from December 1958.

POZZOLANS

Pozzolans may be defined as siliceous or siliceous and aluminous materials, which, while in themselves possessing no cementitious value, will in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. Most of the pozzolans contain substantial amounts of materials other than silica, and it seems certain that during the process of chemical reaction other more complex compounds involving alumina, iron and the alkalies are formed. Ordinarily the reaction of cement with water results in the liberation of some calcium-hydroxide which is soluble in water and contributes nothing to the strength of concrete. In the pozzolan-cement and water mixture, the principal products of reaction are said to be a hydrous calcium-silicate, perhaps monocalcium silicate, which is of relatively low solubility and contributes to strength in concrete.

Researches in the Rihand Dam Research Laboratory were therefore, directed to locate suitable pozzolans for use in the concrete in the dam. Two pozzolans viz. Bokaro fly ash and Bansi clay were found suitable and their effects on concrete were studied in detail. One of the foremost consideration

in the choice of these pozzolans was their cost, which would be about half of the cost of cement at Damsite.

Fly Ash

Fly ash is a waste product of the pulverised coal burning electric power stations and is collected in the mechanical or electrical precipitators. Samples from Kanpur Electric Supply Co., Kanpur, and Thermal Power Station, Bokaro were collected and tested for pozzolanic efficiency. Though the fly-ash from Kanpur was found to be efficient, this source was not considered for supply of fly-ash due to small supplies of 20 tons per day only and the difficulty in quality control. All efforts are now being made to procure our requirement of fly-ashes, i.e. about 100 tons per day, from Bokaro.

Bansi Clay

There are some deposits of a naturally calcined clay near village Bansi, about 26 miles away from Damsite. There are different types of clays and one of these has been found to be a very active pozzolan. Arrangements have been made to procure a suitable crushing plant for pulverising the clay as this pozzolan is expected to be cheaper than Bokaro fly-ash.

Both these pozzolans may replace about 20 per cent of cement by weight without loss in strength of concrete after 90 days. The watertightness of the concrete may also increase by about three times.

"AIR ENTRAINMENT"

The use of air entrainment has permitted an appreciable reduction in the water demand and cement factor in concrete by increaing its workability. The resistance of air entrained concrete to severe weathering is several hundred per cent greater than the non-air entrained concrete.

Non-air entrained concrete contains 1 to 5% entrapped air which is not uniformly distributed in concrete. Bleeding in concrete also creates water channels resulting in more voids when water dries up later on. The permeability of concrete depends directly on the formation of these water channels. Purposefull and uniformly distributed entrainment of microscopic air bubbles of diameter from .003 to .05 inch changes the pore structure of concrete by minimising the bleeding channels. Reduction in water demand and consequently in bleeding and the increased durability has resulted in structures with longer life requiring less expenditure for maintenance and repairs.

Selection of Air Entraining Agent

Detailed tests were carried out in Rihand Research Laboratory to select a suitable air entraining agent. Out of the many brands of air entraining agents in the laboratory, Vinsol Resin NVX, an American product, was adjudged the most suitable in respect of efficiency and cost.

Rihand AEA

Further research was done to develop an air entraining agent from indigenous products. Several raw materials were tried and finally an air entraining agent viz. Rihand AEA has been developed from gum resin obtained by distillation of oleo resin derived from pine wood trees. The chemical composition of gum resin is as follows:—

Resineous acids.	90%.
Resins, esters, and hydrocarbons.	10%.

The presence of large amounts of free resineous acids makes gum resin highly reactive and superior to resin as regards air entrainment. The resineous acids are saponified by processing with caustic soda, and yield the water soluble resin salts.

The constituents of Rihand AEA are therefore gum resin, caustic soda and water; the caustic soda being about 13 to 18% of the resin, and water depending on the concentration of the solution desired.

This air entraining agent was thoroughly tested to investigate its effect on concrete, was found satisfactory in every respect and conforms to ASTM standard specification for air entraining agent. Rihand AEA was also tested in United States Bureau of Reclamation Laboratory, Denver and was found comparable to high quality air entraining agent manufactured in the United States, while the reduction in water requirement was found to be even more than for Vinsol Resin.

Rihand AEA is therefore, being used at Rihand Dam since the comencement of the work and 1.65 crores cft. of concrete has already been placed with satisfactory results.

Air Content in Concrete

The volume of optimum air content in concrete is a moot point but it is almost universally accepted that 3 to 6% air in minus 1½" portion of concrete gives the best results.

At Rihand Dam the decision to use entrained air was based primarily on considerations of workability and severe weathering is not likely to occur. Experimental results on the optimum air content determined in the Rihand Laboratory are shown in figure 9. On these considerations the following air contents in different concretes are being used:—

TABLE 4

Type of mix.	Air Entrained	
	By absolute volume	In—1½″ fraction of mix.
	%	%
Mass Concrete A_6S_{25}	3 – 3.5	4.5 – 5
Face concrete A_6S_{10}	2.6 – 3.1	3.8 – 4.3
„ „ A_6S_{30}		
Concretes A3/2 S_{30}	2.8 – 3.8	2.8 – 3.8
Special Concrete A3/4 S_{60}	2.5	2.5

AGGREGATES

As natural aggregates are not available in Rihand river the aggregates for concrete are produced by crushing the rocks obtained from Makra Quarry and transported to the stock piles by two sets of 2½ miles long rope ways.

This quarry contains predominantly good granite and gniess rocks with small seams of quartz and weathered schists. The granite and gniess rocks are on the whole sound and have the following properties:—

1. Coefficient of hardness. 18.2
2. Resistance to abrasion. good.
3. Compressive strength 15,686 to 22,794 p.s.i.
4. Unit weight. 175 lb/cft.

The maximum size of coarse aggregate has been limited to 6 inches and the aggregate are separated into six fraction of 6″-3″, 3″-1½″, 1½″-¾″, ¾″-⅜″, ⅜″-3/16″ and minus 3/16″. The aggregate fractions ¾″-⅜″ and ⅜-3/16″ and a part of 1½″-¾″ are produced by crushing in hammer-mills which impart improved particle shape. The stock piles are provided with rock ladders to minimise breakage in handling.

A separate article in this issue of the magazine describes the layout and the equipment at quarry and the crushing plant.

Fine aggregate or sand is composed of all particles below 3/16″ size or No. 4 ASTM sieve. As river sand is not available in sufficient quantities and varies in quality it is supplemented by crushed sand obtained by crushing granite rock in two rod mills. The two sands are blended in equal proportions in a Bowl type classifier. The density of the composite sand is 167 lbs. and the fineness modulus is controlled to be 2.5 ± 0.2 with limits of percentages of each sub-fraction. It is of interest to note that about 12 per cent fine i.e. minus 100 mesh particles, have been included in the prescribed grading of sand while the amount of fines should be about 10 per cent from test results as shown in (Fig 11 A). It has been found that about 6 per cent fines are produced in the 4 cu.yd. mixers due to grading action in aggregates and the amount of fines specified has therefore been mentioned as 12 per cent. The general basis of mixing 2.5 fineness modulus was to have minimum water requirement and bleeding as shown in figs. 10 and 11.

Quality Control of Aggregates in field

The tests on aggregates comprise of determination of the (i) density, (ii) voids (iii) internal gradings (iv) oversizes and undersizes, (v) fineness modulus of sand and (vi) moisture contents.

(i) & (ii) Density and voids contained in the aggregates are determined by Techkote air meter. The density affects the volume of aggregates while the voids give an idea of their shape.

(iii) The internal grading of aggregates is determined by sieving each individual fraction over set of screens. The setting of the crushers producing aggregates is checked and altered if necessary when irregular internal grading is detected.

(iv) An aggregate, if varying by less than 1/6th the nominal size, is considered as non-significant oversize or undersize and if varying by more than this is considered significant oversize or undersize. Sieves 7/6th and 5/6th of the nominal sizes are made, and percentages of significant and nonsignificant oversizes and undersizes are determined which should be within the following prescribed limits:—

Table 5

LIMITS OF OVERSIZES AND UNDERSIZES IN AGGREGATES

Aggregate Inches	Oversizes		Undersizes	
	Significant	Non-Significant	Significant	Non-Signification
6-3	Nil	No. limit	2%	10%
3-1½	„	„	„	10%
1½-¾	„	„	„	10%
¾-⅜	„	„	„	15%
⅜-3/16	„	„	„	25%

Adjustment in the mix grading are made to account for the undersizes and oversizes.

(v) Fineness modulus of sand is determined by sieving the sand on a mechanical Rotap sieve shaker through standard A.S.T.M. sieves. The variation between ten consecutive samples is not allowed to be more than 0.2 from the mean. Sand content in the mixes is altered (Fig. 12) if its fineness modulus varies.

(vi) Tests of the determination of moisture content of aggregates are made half hourly and adjustments in batch weights of the concrete mix ingredients are made to correct for the moisture contents. Since mix design computation and mix proportions are based on the assumption of saturated surface dry aggregates, the free surface moisture is determined usually by drying the aggregate on a hot plate and determining the loss in weight. While chan-

ges in moisture contents of coarse aggregates are generally infrequent, the variations in the case of sand are sufficiently important to warrant frequent adjustments in the mix if uninformity of water-cement ratio and the consistency of concrete are to be maintained.

Tests for moisture contents are made half hourly, for fineness modulus of sand every 2 hours, and for oversizes, undersizes, internal grading, density and voids eight hourly.

WATER

The water, to be mixes in concrete is clear and free from injurious oils, acids, alkalies, organic matter, salts, silt or other impurities. The turbidity limit has been specified as 2000 parts per million. Turbidity of mixing water is determined frequently by Jackson's candle Turbidimeter. Water with turbidity higher than 2000 ppm is not used in mixing.

CONCRETE MIX DESIGNS

The following are some average properties of concreates being poured in Rihand Dam:—

Table 6

(See also fig. 13)

Item	Interior	Face	
		Up stream	Down Stream
1. 28 days average compressive strength of 6″ dia × 12″ cylinders p.s.i.	2500	4000	3000
2. Maximum size of aggregates-inches	6	6	6
3. Slump-inches	1½	1½	1½
4. Air content (% air in —1½″ concrete.)	4.5-5	3.8-4.3	3.8-4.3
5. Unit wt. in lbs/cft.	156	156	156
6. Water cement ratio by wt.	0.63	0.43	0.55
7. Water in lbs/cyd.	180	185	180
8. Cement (lbs.per.c.yd.)	286	430	327

9. Aggregates fraction as percent by solid volume of total aggregates.

Mix	6″-3″	3″-1½″	1½″-¾″	¾″-⅜″	⅜″-3/16″	sand (F.M. 2.50)
A_6S_{18}	24	17	14	13	7	25
A_6S_{20}	24	16	14	13	6	27
A_6S_{25}	24	16	14	13	6	27

PRODUCTION OF CONCRETE

The batching is done by weight which is automatically controlled by electrical devices. There is provision for setting each scale of the concrete ingredients for twelve different mixes. By turning the particular mix selector knob and other control switches, any of the twelve mix ingredients can be automatically weighed, mixed and transferred to the concrete conveying bucket. Samples of materials for moisture and other tests can be taken from weigh hoppers.

To ensure that concrete ingredients are weighed within specified limits of accuracy, the scales are checked periodically once a week. Standard sized test weights are used in checking scales. The scale is loaded in increments, making successive use of test weights available until a total weight has been reached which is equal to or greater than the maximum load applied when the scale is in use. If there is evidence that scales are not operating properly during the operation of concreting the objectionable operating conditions are immediately corrected.

Mixing

Four numbers 4-cubic yard Koehring heavy duty mixers, fed through a common automatic surging discharge chute, mix the concrete. The mixing time including the time of charging the concrete ingredients, which is 20 seconds, is 3 minutes. The charging sequence of the concrete ingredients is water, ¾″-⅜″, ⅜″-3/16″, cement, sand, 1½″-¾″, 3″-1½″, 6″-3″ but water is charged such that it is flowing till the last ingredient leaves the weigh hopper.

The ability of mixers to properly mix the concrete is determined by the mixer efficiency tests which are performed frequently.

CONCRETE TESTING AND CONTROL

Sampling Concrete

Samples of mixed concrete are taken for consistency and workability, air content, unit weight and temperature. Arrangements for procuring samples of mixed concrete have been made at the mixing plant under the discharge hopper. The sampling device is shown in Fig. 14. The samples are obtained without any delay to concreting operations.

Consistency and Workability

Consistency and workability necessary for proper placement are measured by the usual slump test and ensure the correct water-cement ratio. Since the repeated batches of the same mix if brought to the same consistency will have the same water content and consequently the same water-cement ratio, provided factors such as batch-weights, aggregates, grading and temperatures of materials are fairly uniform, adjustments for moisture variations in aggregates are made frequently as indicated by moisture tests. It is desirable to maintain as far as possible stable moisture contents in the aggregates, uniform aggregate grading, accurate batching, all in such degree that consistency control within close limits is possible. The subsiding of slump cone concrete indicates the workability of concrete while result of tamping the concrete-cone reveals the consistency of concrete.

Slump tests are made half hourly, and adjustment

in the mixing water is made to assure uniform consistency and workability.

Slump tests are often made at the point of placement also to determine the slump loss in handling of concrete.

Air Content

The air content of concretes is determined every half hourly by means of Techkote air meter which is a direct reading instrument. Since the air content in concrete has an important bearing on the strength, workability and durability of concrete, it is maintained within close limits and tests for entrained air in concrete are done every ½ hour or 45 minutes according to the progress of concreting.

Unit Weight

Unit weight of concrete is determined by air meter for minus 1½″ concrete and in a one cubic foot container for 6″ concrete. This provides a check whether concrete as mixed is according to the design given. The unit weight of concrete is determined theoretically and checked with the laboratory tests. The variations may be due to air-content and unit wt. of aggregates and checks are made to correctly assess and control these variables.

Temperature

The temperature of concrete is measured both at the batching plant and the placement.

Casting Specimens of Concrete

Samples of concrete for casting cylinders are taken every two hour 6″ dia × 12″ cylinders are cast for compressive strength tests; one at 7 days, two at 28 days while one cylinder is tested for some specimens at 3 days and others at 90 days. Extra specimens of some samples are cast for testing after long periods of 6 months, 1 year and 2 years. Beams are cast occasionally for flexural strength tests.

Usually each type of concrete other than mass-concrete is sampled at least once every day for strength tests when it is placed.

The concrete cylinders are covered with wet sacks in the batching plant laboratory for 24 hours, their number marked and then transported to main laboratory and cured in the fog room at constant 70°F temperature and 100% humidity for 27 days. For longer ages the cylinders are kept in separate curing rooms provided with temperature and humidity control.

THE ORGANISATION

The laboratory is headed by an Executive engineer who is assisted by six Assistant Engineers. There are twelve qualified Test and Control Supervisors working under the Assistant Engineers for Test, Research and Control of concrete.

There are two field laboratories, one for testing aggregates located near the aggregate stock piles and another at the batching and mixing plant for testing concrete samples. There is one Assistant Engineer in each shift in charge of inspection and control of production and quality control of concrete. Under him are two supervisors one in each field laboratory to carry out the tests on aggregates and concrete. Two attendants each in the aggregates and mixing plant laboratories and one mistri, are provided to assist the supervisors in sampling and performing various tests.

Besides the two field laboratories for field Control, there is a main test and research laboratory which is well equipped for carrying out the various tests. A Baldwin Universal Testing Machine with a capacity of 400,000 lbs. and equipped with various accessories is used in compressive, tensile, flexural and other tests. Facilities have been provided for making and testing concrete specimens for strength, elasticity permeability, volume change and thermal characteristics. The laboratory has a fog room which is maintained at constant 100% relative humidity and 70°F temperature to provide ideal conditions for moist curing. A well equipped chemical laboratory is provided for chemical tests of cements, pozzolans etc. The laboratory not only provides routine testing of concrete ingredients which serve as a basis for determining and ensuring compliance with specifications but is also responsible for working out concrete mix proportions to secure the maximum value from the available materials.

Acknowledgement

The author is deeply indebted to Shri S. K. Jain, Superintending Engineer & Director, Design and Planning Directorate, Rihand Dam for guidance in writing this article.

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Aggregate Production for Rihand Dam

By A. NAGBHUSAN RAO

RIHAND Project is one of the major hydro-electric projects under the second Five Year Plan. A 300 ft. high straight gravity dam across River Rihand at Pipri (Mirzapur District) will harness the waters of Rihand to generate 105,000 kW. of firm power at Rihand Dam Power Station which will have an installed capacity of 300,000 kW.

The length of the dam is 3,065 ft. and volume of concrete about 6,00,00,000 cft. The U.P. Government called global tenders for the works and the Hindustan Construction Co., lowest tenderer, has been awarded the contract for the entire Dam and Power House structure.

One of the major aspects of the construction is the selection and operation of a quarry for the production of suitable and sufficient quantity of aggregates for the concrete in the Dam and Power House.

INVESTIGATION OF QUARRY

The Department had made preliminary investigations and some quarries were selected on the right bank, about 2 to 3 miles away from the dam. The drilling done by the Department was giving only the amount of overburden and no details as to the nature of rock and the availability in quantity. Further the area was going under submergence as the elevation of the dam was raised. The approach also would be through the submerged area.

Hence, it was decided by the Company to search for a quarry that can be above submergence. Detailed investigations were started both by the Company and the Department and the results were the present quarry area. This quarry is about 3 miles away from dam site and on the left bank. The area of rock plateau is about one mile long and about 3 furlongs wide. Investigations were carried out to ascertain the depth of overburden and nature of rock. Diamond core drilling was adopted. About 26 cores at different areas of quarry were taken to a depth of 100 feet in rock. It was noticed that the overburden i.e. the earth and the weathered rock was from 15 ft. to 30 ft. The total quantity of rock required was about 6,00,00,000 cu.ft. Considering the overburden it was decided to have an area of 6,00,000 sft. and go 100 feet in rock.

The cores showed that good quality granite and gneisses could be had in sufficient quantity. It was also noticed that there were some faults and veins of schist and injection gneiss, hence these were avoided from the areas of operation. Sample of blasted rock and cores were sent to Alipore Test House and the rock was certified as a suitable aggregate producing one.

Removal of Overburden

The quarry overburden removal was started first with labour. All the area cleared of trees and shrubs and the area with minimum overburden and where rock was exposed was tackled first.

Trolly line and trollies were extensively used. D-8 Tractor was also used to doze out the soft areas. Rooter was tried for removal and was successful only in a limited area and that too when the ground was wet. Shovels and Euclid dumpers were also employed where the overburden was heavy. The weathered rock needed drilling and blasting. But the main earth overburden was removed by labour during and after monsoon months when work in the dam site could not be carried out. Thus work was provided always to the labour.

Quarry Development

The opening up of the faces, the number of benches and the number of faces etc., were all worked out in detail. The position of the primary crusher, the shovels, the transport, the approach roads, the drainage, disposal of overburden and the type of drilling equipment and other connected factors governed the development of quarry.

The length of the quarry is about 1100 ft. and the width about 600 ft. average. About 200 ft. of quarry area is bad with too much bouldering area and overburden. This divided the quarry into two sections. So two faces were taken up. In terms of elevation, the

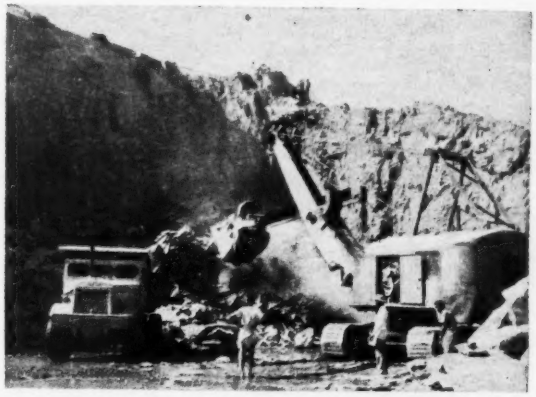


Fig. 1—Quarry face : Shovel loading Euclids.

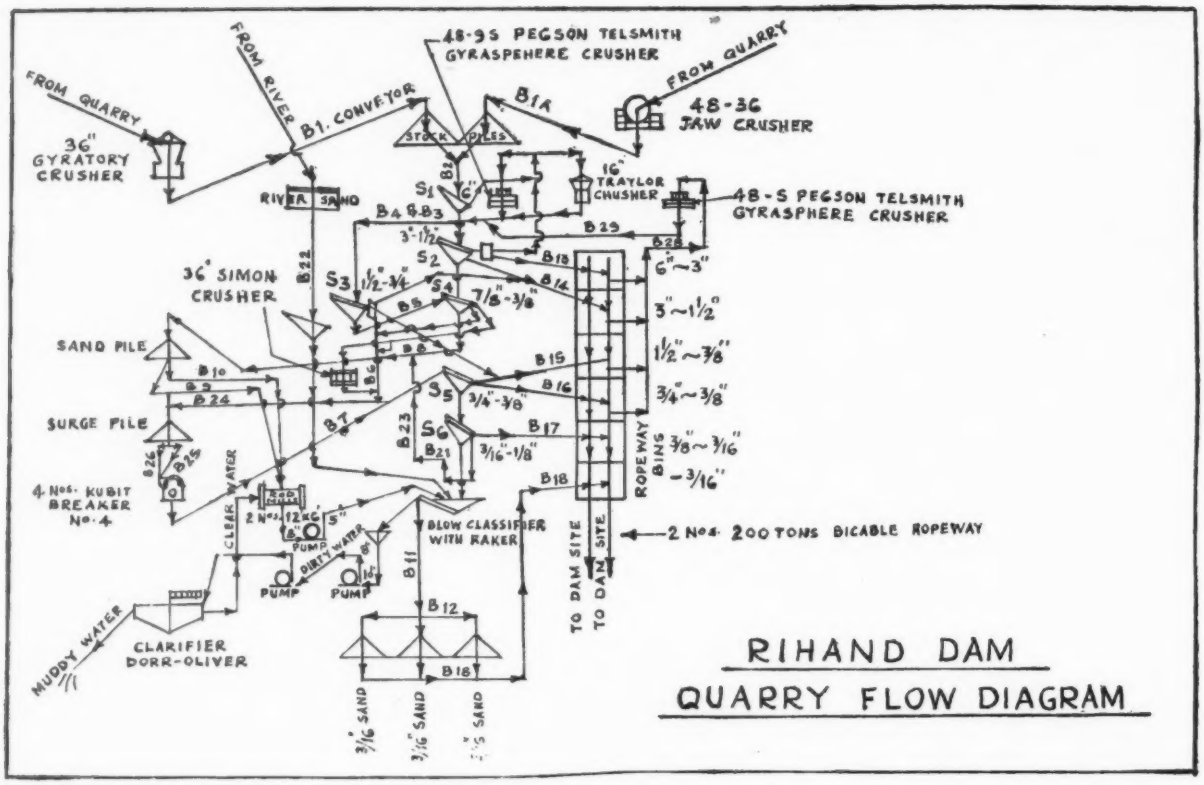


Fig. 2.

quarry top is R.L. 1180.00, overburden about 20 ft. average and so rock R.L. 1160.00. But this is a varying elevation all over.

The first stage of quarry floor was selected at R.L. 1140; the final level being 1080. In the initial stages it was decided to work the quarry in three benches. The final one will depend upon the drilling equipment. It took almost 6—9 months from the first blast on the face to make two good 350 to 400 ft. long, 50 ft. to 70 ft. high faces. The second stage at a level of R.L. 1100 is being opened up. The third stage will be a level of R.L. 1080.00 (or 1070 depending on actual quantity of concrete).

Three approach roads are made to the Primary Crusher which is at the elevation 1142.00 and about 800 ft. from the nearest face. Two-way traffic premix asphalt road with necessary turning area is made from the junction of three approach roads to the crusher.

Drilling & Blasting

The drilling equipment used is the Atlas Copcos Hand-held deep drills and deep drill rods. Atlas Copco make a solid piston wet drill Jack Hammer. The drill steels are "SANDVICK" 7/8" hexagonal rods with detachable 36 mm chisel bit and shank rod sets and extension rods. Three men are needed for each machine while drilling deeper than 30 ft. (Two drillers and one helper). The drilling is wet. About 14 to 18 deep drilling machines work in each shift for two shifts depending on requirements. 10 to 12 ordinary jack hammers work for secondary and boulders in the 3rd shift. Wagon drilling using 2¼" bits is also being adopted. The drilling pattern varies depending on the type of rock and face. Usually, the very unconventional pattern of 8′ spacing and 3 ft. burden is adopted. This pattern was adopted after trial of all sorts of arrangements; the main object being to get a well fragmented heap.

Explosives are blasting gelatine 1" or 1.1/8" dia. strengths varying from 60% to 90% depending on whatever is available from Messrs. Imperial Chemical Industries.

Milli. second short delay detonators No. 0 to No. 6 are used for detonation of all main blasts. Ordinary electric for secondaries. Most of the blasting is by electrical detonation. In very few cases and in overburden, ordinary hand blast is done. Main blasts are done once in 3 days and secondary once a day. Regular maintenance of Jack Hammers are carried out in the workshop. The bit grinding is done on site. Air line lubricators are used along with Jack Hammers.

Air Supply

A central air compressor station supplies air to the drilling, workshop and the crushing plant and ropeways. Four numbers, two stage electrically driven Broomwade make compressors of 512 cft. capacity and two numbers diesel compressors of 350 cft. capacity each make up the station. Now two more Atlas-Copco two-stage 571 cft. capacity each compressors are being added to the station.

Ample receiver capacity is provided. A six-inch main about 1500 ft. long carries the air to the quarry for drilling. The six inch main runs all along the periphery of the quarry and 4″ tappings are taken wherever needed.

Water Supply

For the deep drilling and wagon drilling, filtered water is used. A central tank at elevation R.L. 1300.00 supplies water through a 4″ gravity main to the quarry. The raw water is pumped to quarry all the way from Rihand River (elevation R.L. 630.00). Pumping is in two stages. The main is 12″ dia. At quarry end, the system consists of storage tank, settling tank and rapid sand filters and filter water storage tank and pumping sets for pumping to elevated tank at R.L. 300.

General

The quarry has to supply every day on an average of 6000 tons of blasted rock. This rock must be good and must be well fragmented to suit the two Primary Crushers (36″ Gyratory and 48″ × 36″ Jaw Crusher). The blasted rock must be in good heaps for easy shovel operation. The blasts are to be controlled to avoid excessive throw, of bottom toes and benches. The drilling and othtr operations are carried out in a well organised three shift.

Loading & Transport

The blasted rock from the quarry has to go to the Primary Crushers. The loading is done by 2½ cyd. shovels. There are two diesel Northwests and two Electric Ruston Bucyrus. Two shovels work in each shift. The two 54-RB electric shovels are of Ward Leonard Control type. Supply is 3300 V. Each has a trailing cable 8oo ft. long. Six nos. Euclid dumpers 36BF type with Rock Body and Rolls Royce engines are used for transport. A D-8 tractor keeps the quarry clear after blasts and helps shovel operation.

Crushing Plant & Aggregate Processing Plant

The aggregate processing plant comprises of the Primary Crushers, the primary stock-pile, the secondary crushing screening plant and the sand processing plant.

Design of Plant & Layout

The plant is based on a designed peak capacity of 500 tons per hour and two shifts or sixteen hours operation. This corresponds to a concrete placing capacity of 240 cyds per hour for twenty-four hours.

The Company had in its possession a well-laid out 200 tons/per hour capacity plant is Vaitarna, and

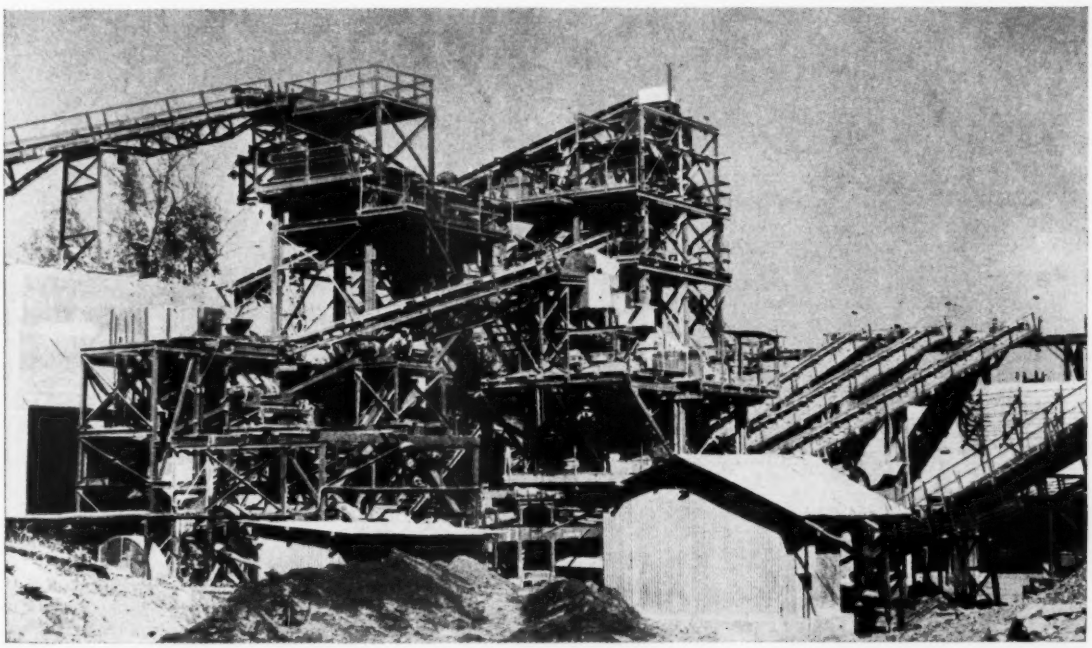


Fig. 3—Crushing and Screening Plant.

Fig. 4.—View of Damsite from top of headmast.

had operated it successfully. The plant at Rihand was entirely planned, designed and erected and put into operation by the Company. It did not seek any outsiders' help. In designing the plant for Rihand, the existing Vaitarna Plant was taken into consideration. Vaitarna quarry was basalt rock whereas at Rihand it is granite. Before finally selecting the equipment, the experimental crushing of Rihand Rock was done at Vaitarna, the laboratories of Pegson Ltd., England, and Allis Chalmers, U.S.A.

It was noticed at Vaitarna, where two wagon loads of Rihand rock were crushed and processed in Gyraspheres and cone crushers that smaller aggregates were flaky and when this was passed through Kubit Breakers, the shape improvement was surprising. The product from the small Jaw Crushers which the Department was working on site also showed that the smaller aggregates were flaky.

Serious thought was given to the heavy wear on the Kubit Breakers as it had to process granite. But considerations of saving in cement and good workable concrete due to good shaped aggregates made use of Kubit Breakers imperative. Wet processing is adopted for finer aggregate processing and sand processing to get a better control as well as to avoid the dust nuisance.

It was originally intended that all sand will be crushed, manufactured and no river sand to be used. The manufactured sand was to be a blend of Kubit Breaker sand and the rod mill sand processed in the classifier. To get sufficient quantity of sand, the Kubit Breakers were to do reduction as well as Kubitising. For reduction they were to be run at higher speeds and the wear was too heavy. Due to the foreign exchange difficulties, the necessary spares for Kubits could not be had. Further it was also considered by the Company and the Department that it was not in the interest of the nation to import these when alternative arrangements could be had. Hence Kubits are used only for Kubitising i.e. improving the shape. The deficiency in the manufactured sand due to this arrangement is made up by using river sand. These entailed some alterations in the plant. Some modifications in the plant were also made to overcome the difficulties experienced in the plant during operation.

Primary Crushing

The Primary Crushing Section which is separated from the Secondary by a stock-pile, consists of:

Two main crushers discharging into a single stock-pile.

1. 36″ Allis Chalmers "SUPER" Gyratory Crusher (At elevation R. L. 1140). This is driven by a 250 H.P. motor, has a capacity of about 350 T/h.

The Euclid Dumpers unload into a masonry hopper and the material is fed to crusher by means of

"ROSS" chain feeder. This consists of four endless chains of 4″ dia running on a drum driven by a variable speed gear driven by an electric motor.

The discharge from the crusher is carried by a 36″ wide conveyor belt to a stock pile 80′ high.

A 20 ton capacity electric travelling crane is provided over the crusher to clear the big stones that jam across the spider and crusher body as well as to do maintenance and concave and mantle changing.

2. 48″ × 36″ Hadfields Jaw Crusher is situated nearer to Quarry at elevation R.L. 1110. This is driven by a 150 H.P. motor. This crusher is fed by a Plate Apron feeder. The discharge is carried by a 36″ belt to the same stock-pile of Allis Chalmers.

Secondary Crushing & Screening

The primary crusher material which is all below about 8″ is drawn through a 36″ wide belt (b2). This is fed by a 500 ton/hr capacity Locker Vibratory feeder. B-1 belt discharges on a pair of Scalping Screens 12′ × 4′ with 6″ sq. opening. – 6″ material is screened on a pair of Double Deck 3″ and 1½″ Screens S2 (12′0″ × 4′-0″). Required 6″-3″ material goes to Ropeway despatch bin on B-13—30″ belt. 3″-1½″ goes to Bin on B-14-24″ belt. Excess 6″-3″ and 3-1½″ are crushed in the above crushers. -1½″ goes to a pair of Double Deck Screens S4 (12′ × 4′) with 7/8″ and 3/8″ deck screens.

From the Secondary Crushers, 16″ Taylor and 489S Pegson, the material goes on to B3 and B4 belts (30″ wide) to a pair of double deck screens where decks are 1½″ and ¾″. + 1½″ – 3″ from this screen can be taken as finished on B14 belt or can be taken for recrushing on the S4 Circuit through B5 belt. 1½″ and ¾″ goes either as finished on B15 belt or goes for kubitising in Kubit Breakers through B6 – B7 or recrushing on S4 circuit through B5 belt. – ¾″ material is taken on B5 belt to S4 screens. 7/8″ material from S4 Screens are recrushed in a 36″ Symon Cone Crusher.

The discharge of Symon and ⅜″ – ⅞″ from the S4 Screens are taken to Kubit Breaker feed stock-pile by B6 and B7 belts. – ⅜″ from S4 Screens goes to Rod Mill feed stock-pile by B8 belt. The Kubit Breaker stock pile consists of mostly – 1½″ ⅜″ material from all the Secondaries. This material is Kubitised in the four Kubit Breakers. These are in two pairs. Each pair of Kubits is fed by a separate 24″ belt by two gates to each belt in the tunnel below the Kubit feed stock pile.

The Kubitised material is taken on a 36″ belt which

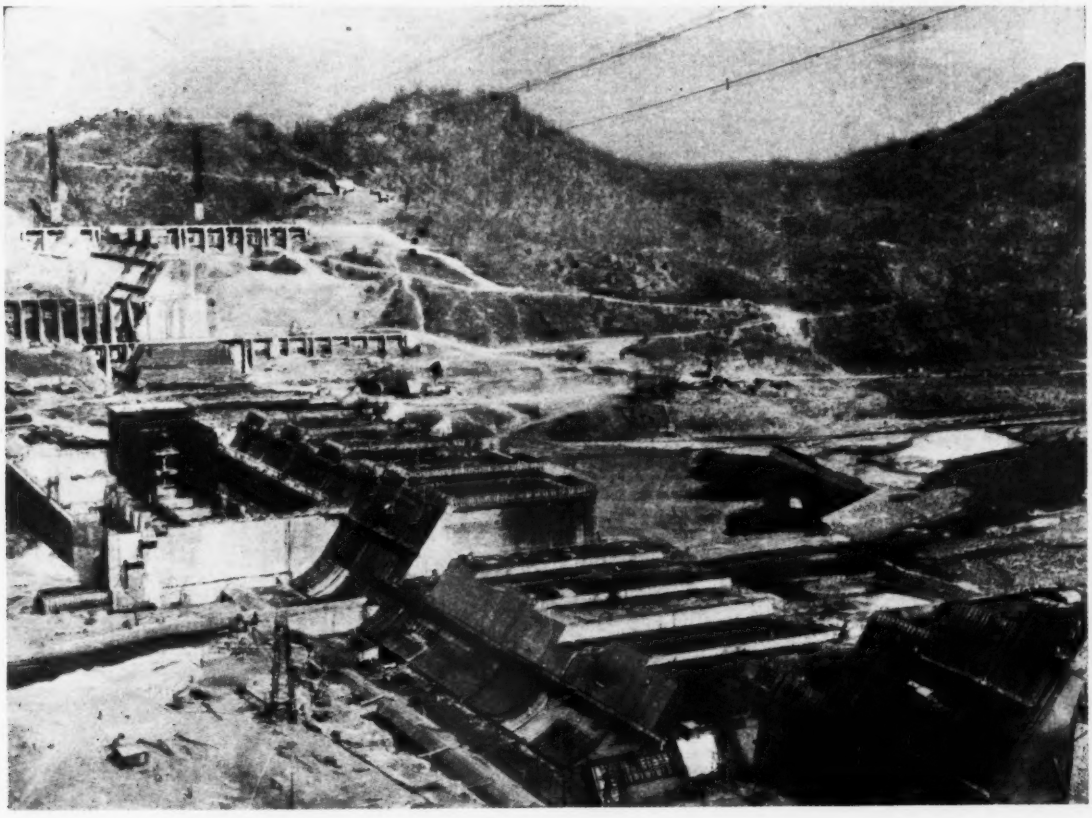


Fig. 5—View of Dam from left bank.

discharges on a pair of 12′ × 5′ wet screens (S5) with $\frac{3}{4}$″ and $\frac{3}{8}$″ decks. $1\frac{1}{2}$″ – $\frac{3}{4}$″ and $\frac{3}{4}$″ – $\frac{3}{8}$″ go on B15 – B16 belts respectively to Ropeway despatch bins. – $\frac{3}{8}$″ goes to the last pair of wet screens S6 12′ × 4′ with 3/16″ and 1/8″ decks. Required quantity of $\frac{3}{8}$″ – 3/16″ goes as finished to Ropeway bin on B17 and 1/8 – 3/16″ and excess $\frac{3}{8}$″ – 3/16″ go through B21 – B22 and B8 to Rod Mill feed stock pile. (– $\frac{1}{3}$″ goes direct to classifier).

The Sand Section

Two rod mills—Marcy & Pegson—of 12′ × 6′ with wet grinding produce the necessary sand. Each rod mill is fed separately by belts from the Rod Mill feed stock pile. The Rod Mill discharge is pumped by one 5″ Wilfly Sand Pump—either on to S6 Screens or direct to 16′ dia Bowl Classifier with 30′ rake. The rake discharges on to a belt B11 which in turn discharges on B12. Belts B11 and B12 form three stock piles about 50′ high to allow for sufficient drainage of wet sand. The drained sand is taken by B-18 belt running in a tunnel under sand drainage stock-pile to the Ropeway bin. The overflow from the classifier is pumped upto Dorr Thickner 100′ – 0″ dia. by two stage pumping using 8″ Wilfly pumps. The thickner or the clarifier is in closed circuit with the sand plant and wet screens. The make-up is added from a storage reservoir.

River sand is brought by lorries from the river bed near the dam site. Samples are taken on the site and analysed and areas marked for sand despatch. This gives the necessary data to control the Rod Mill product so the blended sand is to the requirements. The sand lorries unload into a masonry hopper at the quarry plant. Sand is drawn by a conveyor belt under the hopper and washed down to classifier where it blends with the manufactured sand in the bowl and rakes.

To be able to produce the excess finer aggregates that may be needed for special concrete and also to recrush the excess bigger aggregates a recrushing circuit is introduced.

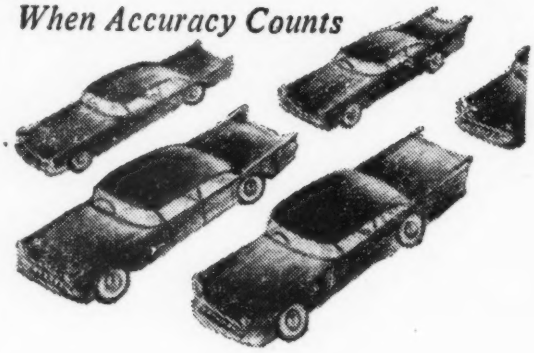
The aggregates 6-3, 3-$1\frac{1}{2}$, $1\frac{1}{2}$-$\frac{3}{4}$ or $\frac{3}{4}$-3/8 can be drawn from the Ropeway Bin on to a belt which feeds a 48″ standard Pegson Telsmith Crusher. This crusher discharges on to B3 belt through belt B-28.

Thus the entire plant is so arranged so as to able to produce the required percentages of different aggregates for the concrete.

The finished aggregates that are taken in a cement concrete bin are transported to the stock-pile at the dam site by means of two independent Bi-cable Ropeways. The capacity of each Ropeway is about 200 tons per hour. The Ropeways work on three-shift basis with regular breaks for maintenance etc .

Thus the Quarry and the Aggregate processing plant—one of the biggest granite aggregate processing plants in the country goes on supplying enough aggregates of good shape and quality for Rihand Dam.

When Accuracy Counts



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Z. 563

The Rihand Dam Rises

By D. M. SAVUR

IN the distant hills of eastern U.P. a concrete dam is fast rising across the gorge of the Rihand River. The dam when completed will form a large reservoir of 8.6 million acre-feet capacity, spreading over 180 sq. miles of area. The power House is located at the toe of the dam on the right Bank. The Power installation will initially consist of six generating units each of 50,000 kW. capacity connected to a 70,000 H.P. turbine. The turbines are to be served by 16 ft. dia. steel lined penstocks installed in the body of dam. Rising 295 ft. from the lowest foundation, the main gravity type dam will contain 2.1 million cu.yds. of mass concrete. The length will be 3064 ft. at crest. The spillway will be provided with 13 tainter gates each 40 ft. × 28 ft. and will be adequate to route a maximum possible flood of 8,41,000 cusecs without overtopping the dam.

The potentialities of the Rihand river—a tributary to the Sone—were known to the country's engineers for long. It was only after the Independence in 1947, however, that the plan for their exploitation and utilization took root. Preliminary operations commenced in 1948. In 1951, explorations had sufficiently advanced at the present site where the river passes through a narrow gorge with well defined outcrops of gneissose-granite. At one time U.P. Government was thinking of executing the project departmentally. But later, global tenders were invited and the work of the construction of the dam and power house was awarded on March 12, 1955 to Messrs. Hindustan Construction Co. Ltd., whose bid of Rs. 17 crores was the lowest.

The nature and the magnitude of the work and the time limit for completion called for heavy mechanical construction plant and a well-planned and coordinated working. Nothing that might seriously interrupt the operations could be left to a chance. Nor was it possible to delay the commencement till all the new plant arrived. Great emphasis was therefore laid during planning on the judicious selection of plant, the construction, and the utilisation of the existing plant till the new plant was available on the site. The existing plant though of a smaller capacity, rendered yeomen service in keeping the wheels of progress moving right from the beginning.

Major decisions had to be taken in selecting the plant, regarding the method of placement of concrete. The choice lay between the "Hammer-head or Revolving Cranes on Trestles" and "Cableways". Almost all the foreign experts were in favour of use of cranes; it involved fabrication and erection of nearly 4000 tons of steel-work in trestles. Procuring this much quantity of steel in a period of few months was almost impossible with the shortage of steel reigning in the country. There again, trestles—it was felt—ran the risk of being exposed to damage due to floating timber or tree trunks passing down the river during floods. On the other hand the span for the cableways had to be 3050 ft. and cableways of such a length with 20-ton load carrying capacity were never tried before. Foreign experts considered the use of cableways beyond 2850ft. frought with risk. The contractor's engineers who had seen 20 ton cableways in operation at the Grande Dixence Dam in Switzerland were convinced of the suitability of the use of cableways and in the special circumstances attending the Rihand Dam, opted in favour of cableways. The Italian firm who had designed the cableways for the Grande Dixence Dam offered to design the cableways for the Rihand Dam conditions.

A good deal of experimentation and testing had to be done in laboratories to finalise the design of the various elements of the cableways. In the final design is incorporated nearly 1500 tons of steel.

The head mast which is 320 ft. high is fixed in position while the tail mast 150 ft. high moves radially on a runway 1015 ft. long. The runway is supported on a rigid structure taken to good foundations.

Much careful planning also went into the other basic operation viz. aggregate production, processing and concrete mixing plant required to meet the demands of a heavy construction schedule.

RIVER DIVERSION

Preparation of foundation for the dam required

Fig. 1—Quarry face: Shovel loading Euclids.

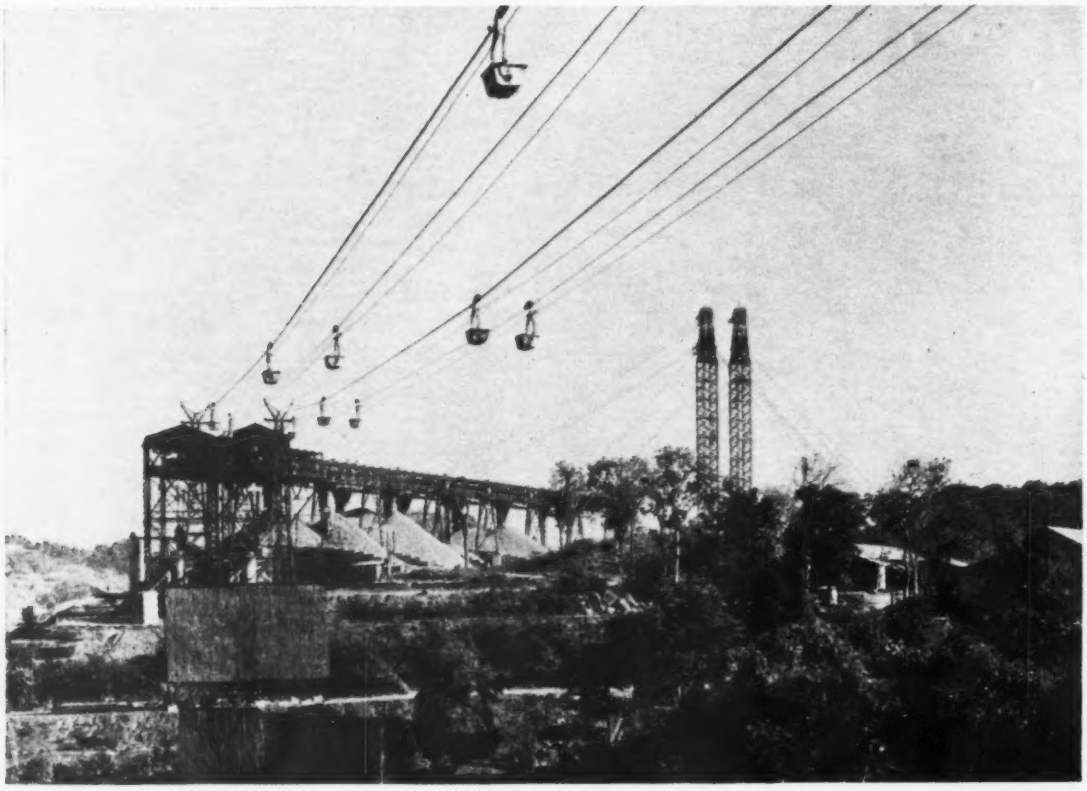

Fig. 2—Ropeways conveying Aggregate to Stock-piles.

removal of nearly 78,500 cu.yds. of overburden and 34,700 cu.yds. of shattered and unsuitable rock. To be able to work in dry space on foundation preparation, the contractor has a three phase river diversion sequence of which two phases have been completed. The diversion system has been designed to cater for dry weather flows upto 8500 cusecs. In the initial phase, the river was diverted through a channel in the right abutment cut specially for the purpose and crossing the dam through monolith 25 and 26. Two masonry cofferdams—one on the upstream and the other on the downstream—were provided to guide the river through this channel. These cofferdams required nearly 16,500 cu.yds. of masonry. By January 1958, all the monoliths of the spill-way were covered up and the power house blocks were raised to a fair elevation. A conduit 12 ft. × 14 ft. was built in Block 37 of spillway for the second phase of diversion and the stream was then conducted into this conduit through another channel flanked by masonry walls. This phase will continue upto a stage when the permanent under sluices in Blocks 38 and 43 are built in at R.L. 710. The lower conduit in B1. 37 will then be closed by a gate on its upstream and the river flows passed through these under-sluices. It is proposed to plug this conduit for its full length by injection grouting using "Cheecol" fluid.

Briefly the concrete is being produced as follows. The concrete aggregates are quarried nearly 3 miles from the dam site on its left bank. At one time quarries were proposed on the upstream side on the right bank. These had to be abandoned in favour of the present quarries as the former were liable to be submerged in the later stages of the work. After stripping overburden upto 30 ft., the contractor has opened out a face nearly 1100 ft. long and 600 ft. wide. The quarry is to be worked in three benches of 30 ft. depth with the floors from R.L. 1170 to 1140, 1110 and 1080. The Primary drilling is being done by hand-held jack-hammers with $\frac{7}{8}''$ extensible steel in pattern of $8' \times 3'$. Wagon drills also drill $2\frac{1}{4}''$ round holes. in a staggered pattern of $8' \times 8'$. Special gelignite of 80% strength is used as explosives.

Down on the quarry floor, the blasted rock is loaded into 15 ton Euclid and dump wagons by four $2\frac{1}{2}$ cu.yd. electric and diesel shovels and is hauled to a feeding platform of primary crusherss. The electric shovels are equipped with Ward Leonord control.

FLEXIBLE PROCESSING FACILITIES

The primary crushing unit provides for two independent circuits feeding into a single 30,000 ton surge pile. A 36″ × 55″ Superior Gyratory Crusher feeds one circuit whereas a 48″ × 36″ Hadfield Jaw breaker feeds the other. The product of the primaries are then reclaimed from under the pile and is fed to a battery of secondary Gyraspheres and cone crusher. One 48″ Gyrasphere, one 16″ Bulldog Gyratory and one 3 ft. dia. cone crusher form the secondary crushing unit. Four Kubit Breakers act as tertiaries to kubitize the $-1\frac{1}{2}$″ material. Six sets of vibrating screens sort out the aggregate into the various grades, namely 6″ – 3″, 3″ – $1\frac{1}{2}$″ $1\frac{1}{2}$″ – $\frac{3}{4}$″, $\frac{3}{4}$″ – 3/8″ 3/8″ – 1/8″ (coarse sand).

On its way from the surge pile, the material undergoes a scrutiny by an electronic metal detector which locates tramp pieces of iron to prevent them from going into the secondary crushers.

The fine sand is processed by two 12′ × 6′ Marcy rod mills and a Dorr Classifier. Natural sand available on the upstream of the dam site is transported about 5 miles to the quarry site. This is blended with the manufactured sand in the Dorr Classifier which functions to separate the – 100 mesh material by floatation and the heavier material is raked out as useful sand. The effluent from the classifier with the floating fines is pumped to a Dorr-Clarifier and is recovered as clear water to be put back in circulation of the floatation system. The sand is stacked in three 55 ft. high piles to drain off its moisture.

The finished aggregate and coarse sand from the respective screens and the fine sand from the stock pile are then conveyed to separate bins by means of a system of conveyor belts. A recirculation circuit with a 48″ Gyrasphere with fine crushing head has been recently introduced to make up any deficiency of smaller aggregates by recrushing the excess in bigger aggregates.

The entire crushing plant is designed for an hourly capacity of production of 500 tons of processed material.

The aggregate from the bins is transported over a distance of 11700 ft. to main stock piles by two bicable ropeways each of 200 tons capacity. The ropeways have a speed of 620 ft. per minute with

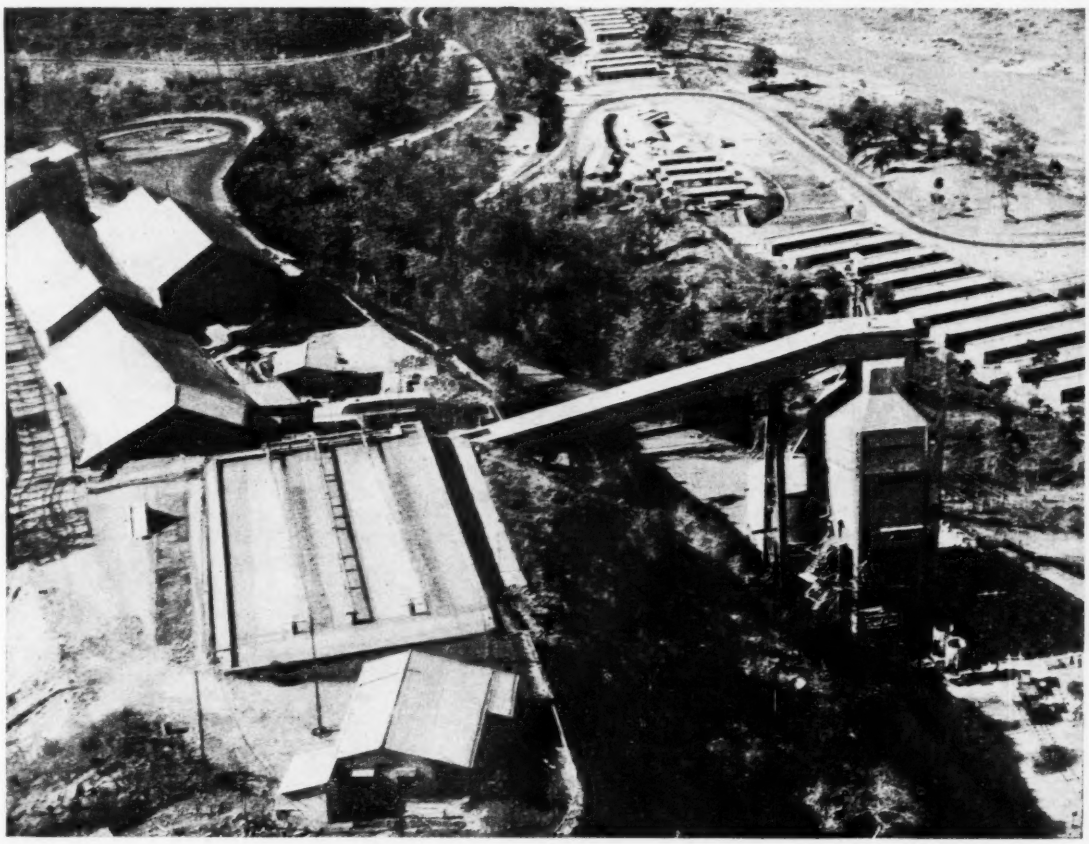

Fig. 3—View of Cooling Plant and Batching Plant.

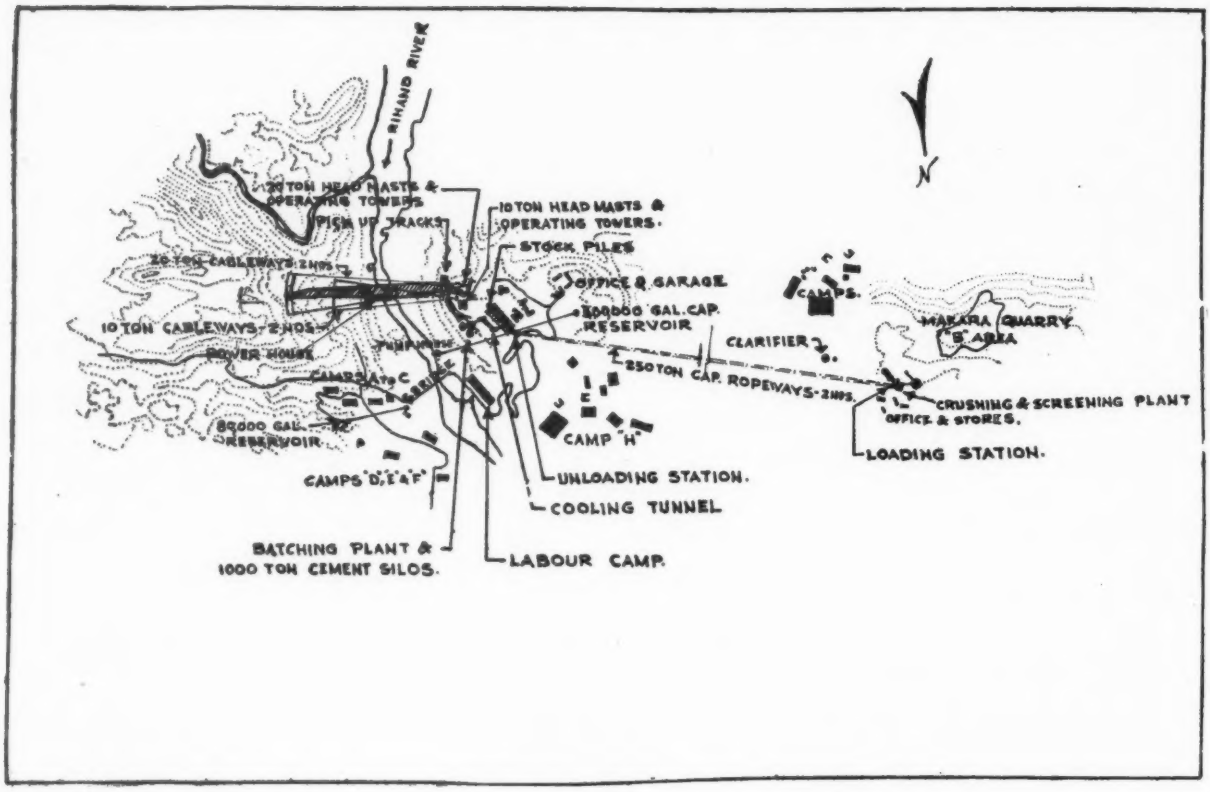

Fig. 4.

buckets of 1 ton capacity spaced at interval of 200 ft. Each ropeway has 38 pylons and two tension gearss. The maximum height of a pylon is 108 ft.

The buckets dump the contents into six 60 ft. high twin stock piles for each size of the aggregates. A recovery tunnel under the stock-piles has a 36″ wide conveyor to transport the graded aggregates into the hoppers at the entry of the cooling tunnels.

PRE-COOLED CONCRETE

The concrete specifications require the temperature of the concrete placed in forms to be not higher than 60° F. This called for a complete refrigeration system for chilling the aggregates, sand and mixing water. The colling of aggregates is done by a process for which patent is held by the contractor in collaboration with Messrs. Voltas Ltd., who have supplied the refrigeration equipment for this job.

In the cooling tunnel, six separate belts for each grade are run in a tightly insulated enclosure 300 ft. long. The four grades of stone are cooled by inundation in 36° F water sprayed on to the conveyor belts. The two grades of sand are cooled by a blast of chilled air at 40° F. The air-blast is designed with particular care so as not to blow off the fines. Suitable churns are also provided on these belts to turn over the sand. The belts have a horizontal run for nearly 200 ft. and further the belts run on an incline with a loop to drain back the water remaining on the stone. The sprayed water is then reclaimed through a channel to a reservoir-cum-settling tank provided with fine screens. The clear water is taken back into the circuit. The cooling of air is done by air-washer which has 8 fans each of 25,000 cfm. capacity. The air-washer utilises 36° F water to produce air at a temperature of 40° F. The water leaving the air-washer is also sprayed on the entry bins of conveyor belts to gain more cooling. The chilled aggregate are fed into insulated bins and then conveyed by 36″ belt to the bins of batching plant. This conveyor again runs in an insulated enclosure where a draft of cold air is introduced by a separate air-washer to prevent any rise of temperature. The 980 ton refrigeration plant employs two 300 ton and one 380 ton carrier centrifugal compressors to chill the required quantity of water to 36° F for spraying on cooling aggregate, air-washers and also as mixing water. A separate sub-cooling unit is installed for further lowering the temperature of mixing water to about 35° F. The resultant temperature of sand and aggregate is about 68° F and 42° F respectively. These when mixed with water

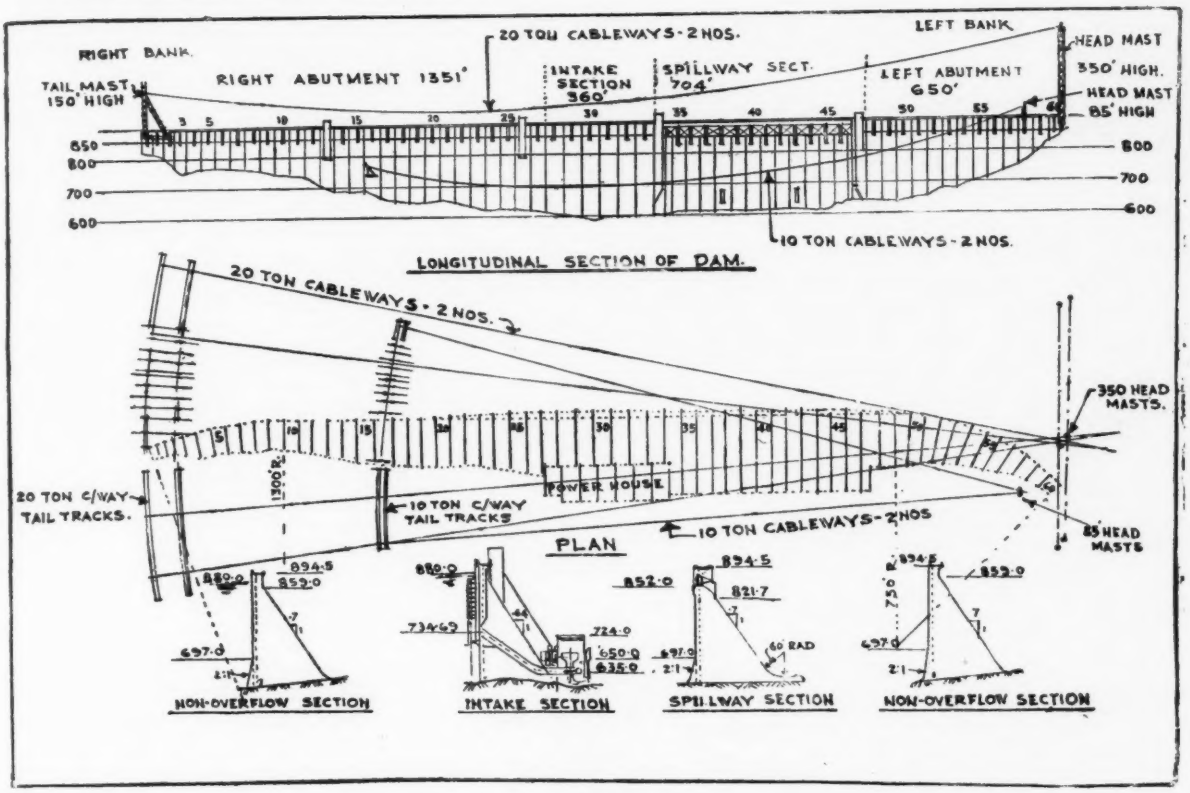

Fig. 5.

at 35° F and uncooled cement gives a concrete temperature at placement point of 60° F, even after allowing for a possible rise of 3° during mixing, transit etc.

Cement for the project is supplied by a Government Factory at Churk and is transported by contractor in bulk in a fleet of 17 ton capacity Gramme trailers pulled by Mack trucks. The cement unloaded from the trailers is taken to the batching plant by a bucket elevator. Extra cement received is stored in two 1000 ton silos. The elevator charges cement either into the cement bin of the batching plant or to the silos which are equipped with aeration nozzles. The cement is reclaimed into the batching plant from silos through the same elevator.

The 980 cu.yds. batching plant is equipped with fully automatic weighing and recording devices. Air-entraining agent known as "Rihand AEA"—a resin caustic soda compound developed by the Test & Research division is used in most of the mixes to entrap 3 to 5% air in concrete.

From April this year, puzzolan to the extent of 18% of cement is being used as admixture in concrete. Fly-ash from Bokaro power house is received in wagons, and transported to a silo of 500 tons capacity near the batching plant. Puzzolan is then elevated into the bin of the batching plant by a separate bucket elevator. Four 4-cu.yd. mixers have been provided to meet the designed hourly rate of 240 cu.yds. of concrete.

The mixed concrete is moved from the batching plant to points directly under the cableway in 8 cu.yd. and 4 cu.yd. buckets placed on wagons drawn by diesel locos on a metre gauge track. The track is equipped with signalling.

CONCRETE PLACEMENT

One of the awe-inspiring operations at Rihand is the concrete placement by the two 20-tons radial cableways spanning 3050′ between the masts. These cableways permit the simultaneous hoisting and hauling of bucket and are powered by D.C. motors with Ward Leonard controls for speed regulation. The position of the bucket is controlled from a control tower located on top of the left bank hill commanding a full view of the dam site where the operator is in direct telephonic communication with the point of placement. The tail tower of each cableway is fitted with four 40 H.P. motors and travels at a speed of 75 ft./min. The speed of 100 ft./min. provided by the makers proved to be rather high by experience and hence was reduced to 75 ft./min. The cableways are designed for an average of 12 cycles per hour and

with 20 hrs. working, the two cableways can pour about 4000 cu.yds. per day. The placement plant also includes two 10-ton. (4 cu.yd) cableways re-erected by the contractor on this job. These cableways being at a lower elevation, mainly concentrate on work on power-house and spillway and yarding. The 4 cu.yd Blaw-knox buckets are operated by compressed air and its opening can be controlled by a regulated quantity of air. The 8 cu.yd Garbro buckets have in addition a high pressure air cylinder which requires filling once in 6 operations.

Concrete mixes are designed in the laboratories on the basis of requirements of strength, slump, max size of aggregate. etc., The mass concrete in the hearting of the dam has a 28 day strength of 2500 lbs/sq.in. Concrete in the dam is laid in five feet lifts in 3 layers of 20″ height. The length of steps is such that concrete not upto grade remains green before fresh concrete is laid on it. The concrete when placed is vibrated by two CP 5190, electric immersion type, 2-men vibrators having $4\frac{1}{4}$″ dia needle with 7000 v.p.m. Sufficient standby vibrators are also provided near the points of placement. Pneumatic vibrators are also kept handy in case there is power failure. After vibration of the pour from a bucket, all loose stones rolling down the unconfined edge are placed on top and the mass is re-vibrated and stones trampled in by men with snow-shoes. To avoid segregation buckets are unloaded not higher than 5′. In difficult corners and heavily reinforced sections like draft tube floors, slabs, columns etc., elephant trunk and flexible rubber chutes are fitted to the bottom of buckets. Bleeding water is removed by sponges.

Cantlever steel forms for 5 ft. high lifts are used for shuttering. At some locations on the upstream and down-stream faces, forms are used with high tensile tie-down rods and anchors to provide rigidly and ensure precise surface profile The downstream face of the spillway and the bucket was being treated initially with vacuum process for improving its resistance to wear resulting from floods passing over it. This process was later abandoned as the results were

(Continued on page 72)

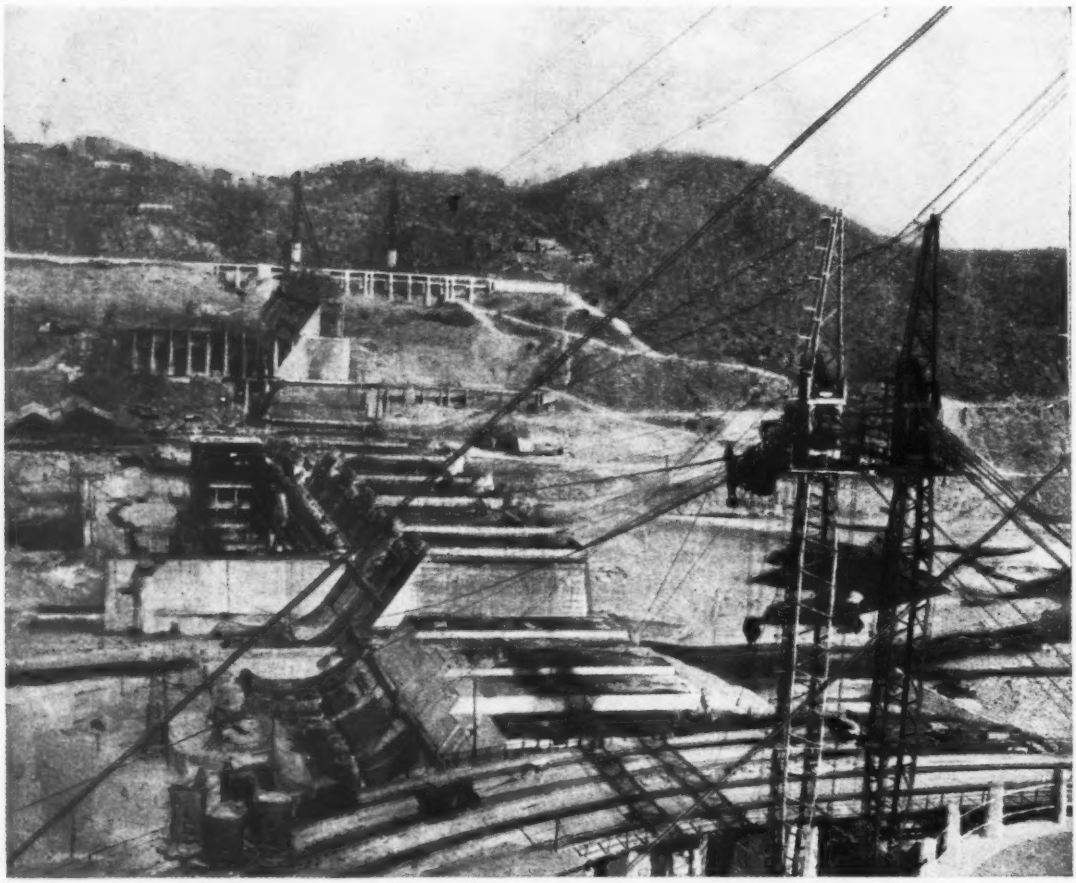

Fig. 6—View of Damsite from left bank hill.

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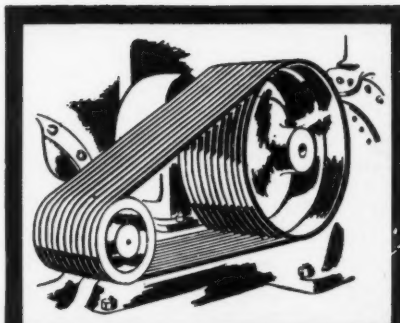

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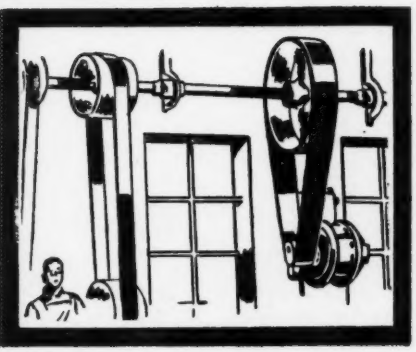

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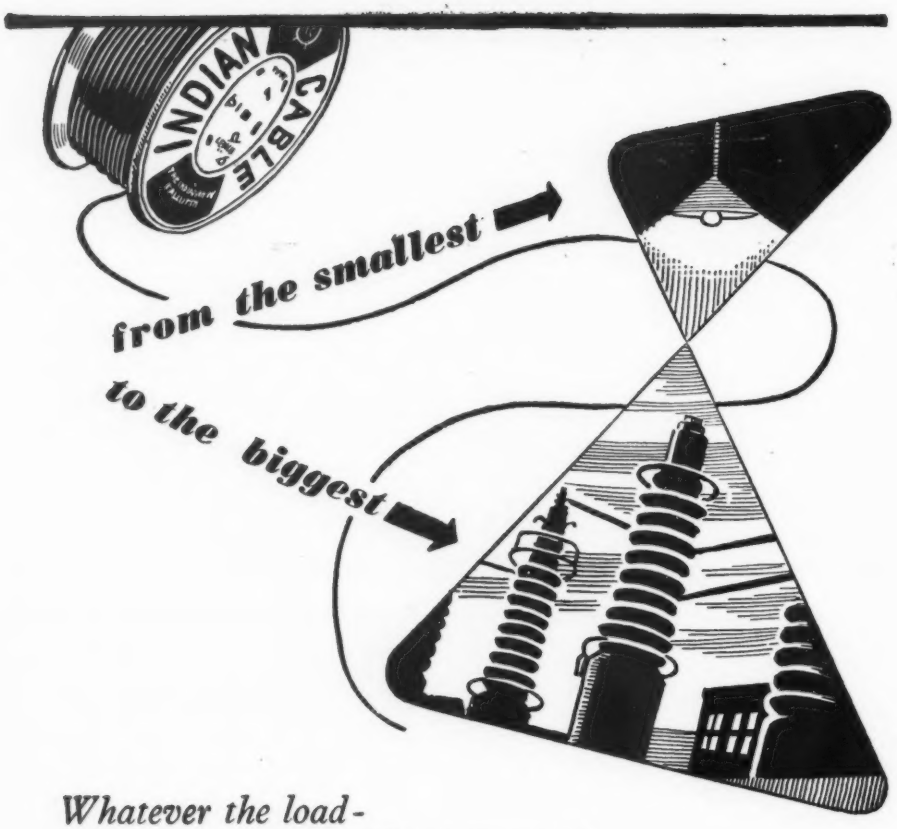

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Power Generation and Unit Control at Rihand Power Station

By LAKSHMAN DEV

THE hydrological studies of power potential of Rihand River over a period of 50 years indicate that firm power to the extent of 1,05,000 kW may be continuously available from the reservoir barring periods of dry cycles. The frequency of these dry cycles is very difficult to forecast but it is anticipated that these may occur once in 50 years or so. In such a situation steam backing would obviously be a necessity. All these considerations as also the interconnection of the Rihand Power Station to a small power potential at Obra 20 miles downstream, with the neighbouring grids of D.V.C. and Madhya Pradesh and the extension of the net-work to the thermal stations at Lucknow, Kanpur vis-a-vis interconnection with the Sarda Grid have been kept in view in planning and designing.

Though opinions may differ as to the number of units that need have been installed for a continuous power potential of 1,05,000 kW, but based on the experience of similar developments in our country and abroad it is anticipated that in years to come Rihand Power Station will get connected to a big thermal source, the latter acting as the base and the former as the peak load station. It will be a combination of thermal and hydro power plants that will provide opportunity for fullest exploitation of the power potential of the Rihand River for the industrial development of the State.

The generation of power from the reservoir will be possible between EL. 880 to EL. 773. Keeping in view that the Rihand Power Station may be required to serve heavy industries as also exchange power with Girds 200 to 300 miles away as also provide power for railway electrification, the control of machines and their operation etc. has been very closely co-related to provide continuity of supply and restoration in the event of breakdowns within minimum time. The importance of aluminium factory and railway electrification loads needs no mention and it may suffice to say that these loads want dead continuity of supply and quick automatic restoration in case of faults.

CONTROLS

Briefly the following controls are being provided on the machines and other apparatus at this Station:

(a) **Unit control:**

The scheme of control in the machines is briefly indicated in sheet No. 1 and is described as follows:

(i) *Dewatering and Drainage Control*

The draft tube and the penstock in a power station require casual dewatering while the top cover of the Machines may require continuous drainage of the leakage water from runner seals and the turbine shaft glands. In addition to the above, at Rihand Power Station the drainage from the dam galleries is also lead to the power house drainage system.

The following pumps are being provided.

(a) 3 – 3000 gallons/unit
(b) 2 – 300 gallons/unit

All these pumps, except one located at EL. 684, are automatic start/stop at predetermined water-levels, through a system of low voltage electrodes adjusted for contact with water surface in the sumps at various predetermined levels and energise/de-energise the operating relays of the pumps.

The operating voltage of the electrode is 50 volts, 50 cycle single phase.

The battery of dewatering pumps provide ample capacity to cope with anticipated drainage from all sources, while on the turbine top cover of each machine a second pump as standby is provided. In addition to automatic control there are local controls on each pumps. The starting and stopping of pumps is flashed on the turbine guage panel which is constantly attended. Further in the event of water level continuing to rise to a danger mark for what-so-over reason, a system of 50 volt battery operated electrodes is provided for alarm. In such an eventuality the reserve dewatering pump at EL. 684 will be brought into service or the standby drainage pump in the top cover of the machine will be switched into the circuit as the case may be.

(i) *Temperature Control*

The important parts of the unit requiring continuous vigil against temperature rise are:

(a) Turbine guide bearing,
(b) Generator combined thrust and guide bearing,
(c) The stator and rotor of the generating unit,
(d) Main step-up transformer.

The temperature control is exercised through the medium of water as a coolant and the measurement from time to time with provision for alarm and even shut down in case of temperature shooting the prescribed mark.

The source of cooling water supply in each case is the penstock, but the water is passed through the strainers before letting in the apparatus. The outlets are discharged direct into the tail race through a non-return valve as protection against back-water. The turbine guide bearing is in oil bath with water passages in the body of the housing. Temperature of the inlet and outlet water as also the body temperature of the bearing are directly indicated on the turbine gauge panel. In the event of the temperature rising a predetermined limit, there will be alarm in the turbine and the control rooms followed by automatic shut down of the machine. Generator thrust and guide bearings are provided with circulating oil pumps forcing oil through a heat exchanger in a closed system. One A.C. and one D.C. pumps are provided, the former is for continuous operation while the latter is for emergency standby and is automatically brought into the circuit on failure of the former. Temperature measurements of the oil bath, of the thrust and guide bearings are directly indicated on turbine gauge panel. Thermostats are embedded in the body of the thrust and guide pads for this purpose. In the event of low oil flow in the circulating system there is an alarm in addition to the standby D.C. pump being switched in to the circuit. If the trouble continues and the temperature of oil and the bearing registers a rise then the machine is automatically shut down on reaching the set limit. It may be mentioned that in the oil flow circuit there is a weir which tends to pond up oil in the thrust bearing such that it can allow running of the machine for about 15 minutes or so without carbonisation.

The alternator is self-ventilated by fans attached to the top and bottom of the rotor. Eight air coolers are mounted tangentially on the stator frame at suitable points. The fans on the rotor draw air from above and below and through the air gap force it in the ventilating ducts of the stator frame into the coolers in a closed system. The water supply to the coolers is through a two-way valve permitting reversal of flow as a measure of automatic descaling of the tubes. Measurements of temperature is made on the air inlet, water inlet and outlet and the body of the stator at a number of points. In case of stator, resistance temperature detectors are embedded at 12 selected points evenly distributed on the 3 phases. There is provision for alarm on generator air temperature. If it continues to register increase to the set limit, the machine is automatically shut down and brought to rest.

The temperature of rotor pole faces is not being measured and the outlet air temperature is being depended upon in this connection.

The main power transformers at Rihand are 60mVA, oil forced & water cooled. Flow relays on the water and oil circuits are provided for automatic shut down of the machine in case of low flow. There is provision for temperature measurement in the transformer oil and is arranged for initial alarm and subsequent shut down of the unit if the temperature exceeds the set limit. It may be mentioned that the temperature though measured in oil is arranged to indicate the transformer winding temperature.

There are other parts of the apparatus in the power house such as station and unit transformer where only local indication of temperature is considered adequate.

(iii) *Humidity Control*

To prevent condensation in the generator thermostat controlled heaters are automatically brought into circuit. These thermostats are set to maintain the generator temperature always above the dew point.

(iv) *Air Compressor system*

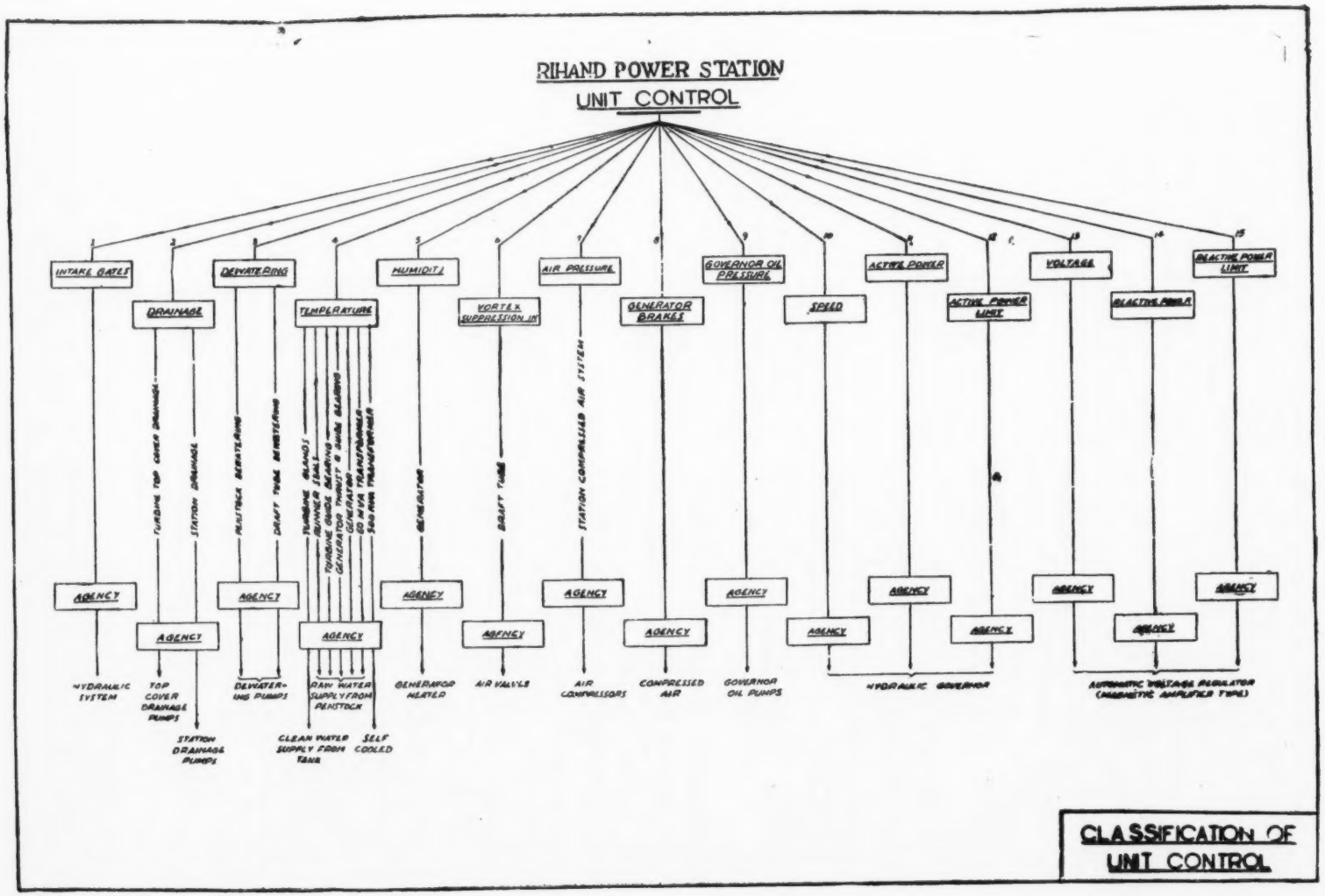
A 300 lbs per sq." air compressor unit having two compressors is provided common for all machines in the Station. Any of these compressors can be set to operate manually or automatically. Each can be set to a different range of pressure. Provision is also made for alarm in case of low pressure. This system will supply compressed air to the governor oil pressure receivers and the generator air brakes.

(v) *Generator Air Brakes*

Compressed air supply from the air receiver is fed to a bus through a stop valve, pressure reducer, air filter and safety valve. It is led to the air brakes of each unit through isolating valve, solenoid operated valve and expansion cylinder. Solenoid controls on the air circuit of the brakes can be energised manually or automatically by master shut down relay. In both the cases the circuit is interlocked with the aux. contact on the 132 kV circuit breaker, auxiliary contact in the gate switch and aux. contact on the under voltage relay. Breaker aux. contact closes when the breaker is open, guide vane limit contact closes when the guide vanes are closed and the under voltage contact closes when the speed falls below 50%. Thus the breakes cannot be applied unless the machine is disconnected from the busbars, guide vanes are closed and the speed of the set is below 50%. Lamp indication for 'Air brakes ON' is flashed on the turbine gauge panel.

(vi) *Governor Oil Pressure Control*

The governor system comprising of actuator, pilot servomotor, distributing valve and guide vane servomotor, uses oil at 270/300 p.s.i. as a media of control. An oil pressure receiver fed by two oil pressure pumps supplies oil to the above system. The 30 h.p. motor driven pumps supply the oil from the sump


RIHAND POWER STATION
UNIT CONTROL
1 INTAKE GATES
2 DRAINAGE
3 DEWATERING
4 TEMPERATURE
5 HUMIDITY
6 VORTEX SUPPRESSION IN
7 AIR PRESSURE
8 GENERATOR BRAKES
9 GOVERNOR OIL PRESSURE
10 SPEED
11 ACTIVE POWER
12 ACTIVE POWER LIMIT
13 VOLTAGE
14 REACTIVE POWER
15 REACTIVE POWER LIMIT
TURBINE TOP COVER DRAINAGE
STATION DRAINAGE
PENSTOCK DEWATERING
DRAFT TUBE DEWATERING
TURBINE GLANDS
TURBINE GUIDE BEARING
TURBINE THRUST & GUIDE BEARING
GENERATOR
60 MVA TRANSFORMERS
500 KVA TRANSFORMERS
GENERATOR
DRAFT TUBE
STATION COMPRESSED AIR SYSTEM
AGENCY
HYDRAULIC SYSTEM
TOP COVER DRAINAGE PUMPS
STATION DRAINAGE PUMPS
DEWATERING PUMPS
RAW WATER SUPPLY FROM PENSTOCK
CLEAN WATER SUPPLY FROM TANK
SELF COOLED
GENERATOR HEATER
AIR VALVES
AIR COMPRESSORS
COMPRESSED AIR
GOVERNOR OIL PUMPS
HYDRAULIC GOVERNOR
AUTOMATIC VOLTAGE REGULATOR (MAGNETIC AMPLIFIER TYPE)
CLASSIFICATION OF UNIT CONTROL

through non-return valve to the oil receiver. The oil pressure is by-passed through a valve and opens to the sump when the pressure is 300 p.s.i. and closes when the pressure is 270 p.s.i. The pumps are to run continuously and pump oil to the receiver from 270 p.s.i. to 300 p.s.i. Out of two pumps one is main and the other standby operating automatically by a pressure relay. Either of these pumps can be selected for this purpose. Indication of the running and stopping of the pumps is flashed on the T.G. panel. There is provision for alarm when the standby pump gets into circuit. If the pumps are not able to maintain pressure then the oil pressure relay initiates alarm followed by shut down of the machine.

(vii) *Active Power Limit Control*

This is accomplished by limiting the maximum guide vane opening through a load limiter motor on the governor actuator which can be controlled both locally and remote from the control desk.

(viii) *Speed Control*

A 1.2 kVA alternator with permanent magnet poles is mounted vertically above the main alternator for supply to the synchronous motor driving the governor pendulum, the tachometers (one on turbine gauge panel and other on control desk), the overspeed relay and the under voltage relay.

The synchronous speed of the machine is 150 r.p.m. The governor mechanism provides two important devices of speed variation:

(a) Permanent Speed variation,
(b) Speed Control gear.

The former in case of Rihand machines permits adjustments between 0-5% and provides for static speed characteristic. The later device permits shifting of the above characteristic on the frequency scale by $\pm$ 5% as may be warranted by load conditions. With the above two devices, it will be possible to maintain frequency of the station within close limits of the declared figure of 50 cycles and also achieve desired load sharing between the machines. The permanent speed adjustment is not a matter of day to day control but is set initially when the unit it put into service. It can however be changed later if need be. The speed control is however subject to constant adjustment according to load directly from the control desk.

A change in load causes disbalance between electrical torque of the generator and the mechanical torque on the turbine leading to change in speed. The disbalance would continue till the latter is adjusted to suit the load requirements.

This involves opening and closing of guide vane apparatus by the governor mechanism through a set of oil pressure relays. The instantaneous variation in speed is minimised by the high inertia constant of the unit which in case of Rihand is 4.00. The WR^2 of the machine is $42.79 \times 10.^6$ lbs. ft^2. The governor mechanism also incorporates return motion gears at various stages to promote stable operation of the machine i.e., free from hunting as far as possible. The other devices associated with the governing of the units are:

(a) Active power control,
(b) Active power limit control.

These form part of the actuator mechanism and are capable of adjustment according to the operational conditions from the control desk. The actuator mechanism also includes guide vane position and guide vane limit position transmitter for indication on the turbine gauge panel and control desk.

The foregoing controls on the machines permit flexible operation under varying conditions of load and reservoir level.

EMERGENCY CLOSURE

The emergency closure of the turbine by the governor can be initiated by manual press of the emergency buttons on the control desk, turbine gauge panel and field suppression cubicle or automatically by the generator transformer protective devices. In the event of underspeed and pendulum drive failure an automatic emergency closure is brought about by a pressure relay in the actuator mechanism.

VORTEX SUPPRESSION IN DRAFT TUBE

Model tests in the hydraulic laboratory have shown the formation of a whirling vortex in the draft tube under certain load and head conditions. The suppression of this vortex in the laboratory was achieved by the admission of air through a crosspiece in the draft tube below the runner. Similar arrangement is being provided at site and the admission of this air will be regulated by the governor according to gate opening. The suppression of this whirling vortex is essential as it may otherwise lead to intensive cavitation as also vibration in the machine. In addition, arrangement is being provided for admission of air through the top cover of the machine through air sniffing valves in the event of sudden closure of the machine.

INTAKE GATE CONTROL

The intake gates are oil pressure operated and are controlled by means of various pressure relays starting and stopping push buttons and other switches from the intake gate control panel located in penstock operating gallery in the dam. The gates are designed and interlocked to operate only under conditions of equalized pressure of water on both sides. That means filling the penstock line before gate operation. The gates are held in position by oil pressure below the piston in the operating cylinder and a continuous restoring cycle is maintained by pressure relays. Arrangement is also provided for alarm if such restoring cycle fails. The emergency closure of the

overspeed and overvoltage relays in the power station. It is also possible to hold the gate at any position of its operation and even reverse the cycle. The indication of gate position is flashed on the turbine gauge panel.

A.V.R. AND FIELD SUPPRESSION

The excitation system on the machine will comprise of control exciter coupled to a high frequency motor generator set, main exciter and field suppression equipment. The voltage of the machine can be controlled both manually and automatically. The limits for the later are $\pm 10\%$. Manual control is through a motorized rheostat in the main exciter field. Automatic voltage control is effected by comparing the rectified generator voltage with a reference circuit voltage in the control circuit of the voltage regulator and the error signal being amplified by means of magnetic amplifier, which in turn effects the control exciter boost or buck field. The output of the control exciter is fed to the field of the main exciter. The voltage regulator is arranged for both parallel running and line drop compensation. In case of failure of the voltage regulator an auto/manual change over relay is provided in conjunction with a follow-up circuit in the voltage regulator which ensures that the voltage of the machine is maintained to the previously set value. The voltage to which the A.V.R. will automatically regulate can be set from the control desk by operating the motorized rheostat in the control circuit of the A.V.R. Undervoltage and fuse failure relays are also provided as protective devices. The A.V.R. circuit is also arranged to have control over the minimum excitation limit of the machine and does not allow the excitation to fall below a predetermined level which will make the operation instable.

SYNCHRONOUS CONDENSER OPERATION

With a view to work the 55 mVA Rihand generators as synchronous condensor in an interconnected grid system at some future date provision is being made for such an operation being possible. It would require suppression of the water in the draft tube and a system of alarm in this connection. The necessary embedments in the foundations as also provision of space for mounting of the necessary equipments has been made.

The above is a brief outline of the controls that are being provided at the machines at Rihand Power Station. It is hoped that the above will ensure continuous power generation as also full safety of the machines in various operating conditions. With a judicious distribution of load as the hydraulic conditions may require from time to time the five machines at Rihand Power Station are expected to generate 919.8 million units per year.

(Continued from page 68)

THE RIHAND DAM RISES

inconsistent and the water extraction was non-uniform.

PROGRAMME AND PROGRESS

Despite delays, the Rihand Dam construction is on schedule. The Suez canal closure towards end of 1956 delayed the arrival of some component parts of the batching plant and cableways and commencement of concreting operations had to be postponed by a few months. The work also received a serious set back when the river brought unexpected floods in Febuary 1958, flooding the entire area of power house and spillway. The concractor, however, braced up to face the situation and closure of work did not last for more than 12 days.

With the first bucket placed on April 5, 1957, the concreting operations continued till end of August by two 10-ton cableways only. During the season the crushing, batching and mixing and other plant passed through an initial shakedown period and the system was harmonised. Nearly 41,000 cu.yds. were poured by the close of the season.

The concreting work recommenced in October 1957, with one 20 ton cableway coming into operation. The second 20 ton cableway was commissioned in January 1958 and the daily progress stepped up to 2600 cu.yds. The maximum pour in a month was 65,000 cu.yds. The total concrete at the close of season in October was 6,05,000 cu.yds. A short break of 3 weeks was taken in October 1958, for routine check up and maintenance on the plant. The construction schedule for the next two seasons was keyed to the requirement of storing waters of 1960 run-off behind the dam upto R.L. 820. The tempo of work had therefore, to be accentuated. The concreting work was resumed toward end of October 1958. The progress stepped up both on the dam and power house. The daily concrete averaged about 3500 yds. The max. concrete pour in a shift of 8 hrs. was 2128 cu.yds The max. concrete during the month of March 1959 was 1,00,000 cu.yds. The plant was shut-down from middle of September for annual maintenance. The total concrete at the time of writing is 1.52 million cu.yds.

The Power house structure was ready for the erection of the gantry cranes towards the end of April. The draft tube liners are already in place. The scroll casing of the first unit is also installed. The erection of scrolls of the next two units is in hand. The steel work of the power house structure is also nearly finished. The work in the power house is now in full swing.

Power House Design

By M. P. TYAGI

RIHAND Dam is primarily a power project. The water stored in the reservoir formed as a result of the construction of the dam will be utilised for production of power, by building a powerhouse at the toe of the dam.

GENERAL DESCRIPTION OF THE POWERHOUSE

The powerhouse will have an ultimate installed capacity of 3,00,000 kW., but only five units of 50,000 kW. each are being installed at present and space is being left for future installation of sixth machine of the same capacity. The turbines will be of Francis type coupled with the usual vertical shaft alternators. Water will be carried to the turbines through six separate penstocks of 16 ft. diameter each embedded in six blocks Nos. 29 to 33 of the dam.

The powerhouse building will consist of six unit bays and a service and erection bay. Each of the bays will be 60 ft. long. These will be located at the toe of blocks 27 to 33 of the dam on the right bank of the river. Each bay will be structurally independent of the adjacent bays and has been separated from each other by the provision of contraction joints upto the level of the draft tube, where the daily or seasonal variations of temperature will be very small, and by one inch expansion joints above the draft tube where temperature variation will be quite appreciable. These one inch expansion joints extend throughout the superstructure upto the roof level and would also prevent the transmission of vibration of one machine to the other. The powerhouse will also be separated from the dam by a similar one inch joint so that the deflection of the dam blocks may not affect the powerhouse. Expansion joints have also been provided on the penstocks near the points where they cross the expansion joint between the dam and the powerhouse.

The powerhouse will have two floors. The turbine floor will be at EL. 650.00 and the generator floor at El. 663.00. At these very elevations, some longitudinal galleries have been located between the dam, and the powerhouse. One of the galleries at EL. 650.00 will serve as the cable gallery while the other gallery will serve as a passage. The galleries at El. 663.00 will house the unit boards and other panels as well as some stores. Galleries have also been provided at El. 622.00 and 602, the former mainly for the inspection of draft tubes and expansion joints of the penstocks and the latter for dewatering the draft tube. The service and erection bay will have three floors, one at EL. 650.00 which will house oil and other stores and oil purification equipments, the second at El. 663.00 which will house a workshop and the third at El. 683.00 which will serve as the erection floor for the machines. The power station will be served by two electric overhead travelling cranes each having a lifting capacity of 90 tons.

The control building is located at the toe of the dam adjacent to the powerhouse as a separate block. This will be having three floors, the lowermost floor at El. 673.00 will be a basement floor for the cables. The intermediate floor at El. 684.00 will be the main control room. The topmost floor at El. 699.62 will form the offices etc. The switchyard has been located outside and in front of the control building.

Figs. 1 and 2 give the cross-section and plan of the powerstation.

DESIGN FEATURES AND DETAILS OF THE POWER HOUSE

The general design features and details of the various component parts of the powerhouse structure and its appurtenances are discussed briefly in the following paragraphs.

(a) Draft Tubes and Tail Race :

The draft tubes are of the elbow type. The vertical centre line of the draft tubes is at chainage 218 ft. from the dam axis whilst the exit ends are at chainage 273 ft. The velocity at inlet of the draft tube would be about 27 ft. per second and would reduce to about 5½ ft. per second at the exit end. The draft tubes are lined by steel liners upto a point where the velocity gets reduced to 15.75 ft. per second. The liner is of electrically welded construction and is made up of ⅜″ thick steel plates. Thirty two holes were provided in the bottom of the liner at 3 ft. centres for mortar grouting under the liner, after embedment in concrete, so as to ensure the perfect contact of liner with concrete. Each draft tube has been divided into two parts after the bend by means of an intermediate pier. The noses of these piers are also lined by steel liners. The shape of draft tubes is shown in Fig. 3.

A longitudinal gallery running near the foundations of the powerhouse upstream of the draft tubes and sloping from elevation 602.5 to 601.5 will serve as drainage gallery for dewatering the draft tubes. All the draft tubes are connected to this gallery by means of 12 inch diameter pipes with valves which can be operated from the gallery at elevation 622.00. Com-

pressed air connections have been provided in these 12″ dia. pipes to help in washing out silt from them. The dewatering gallery will conduct water to a sump situated in the erection bay with its bottom at elevation 599.5. Two dewatering pumps of 3200 gallons per minute capacity have been provided in this sump. The pumps are automatic and will start working one after the other when the level of the water in the sump rises above predetermined elevations. All seepage drains from the powerhouse and the dam are also conducted to this dewatering gallery and two additional small pumps of 430 gallons per minute capacity have been provided in the sump to pump out the small quantity of seepage water. These pumps have also been arranged to work automatically at lower levels of water in the sump, whilst the bigger pumps will operate only when the draft tubes have to be dewatered. The motors for these pumps have been located at elevation 650.00.

All these pumps discharge into a common 2 ft. dia. discharge pipe which lets out the water into the tail race at El. 634.00. Non-return valves have been provided on all pumps to prevent any possible back flow of water from the tail race during floods.

Another sump has been provided in the spillway retaining wall which has its top at elevation 683.00. The draft tube dewatering gallery extends into this sump. A third pump with a capacity of 3200 gallons per minute has been installed in this sump as a standby for emergencies. This pump will be manually operated and its motor will be located at elevation 683.00 which is above the highest flood level.

The exit ends of the draft tubes could be closed by means of gates which are handled by a gantry crane travelling on the downstream deck of the powerhouse at elevation 663.00. Two gates will close the two openings of one draft tube. Each opening is 20′ × 15′-2″. Each gate will weigh about 13 tons. The gantry will have a lifting capacity of 20 tons. The gates will be lifted under balanced head of water, and a filling line has been provided for the purpose. Dewatering of penstock will also be done through this filling line which can conduct water from the penstock to the draft tube. Three sets of gates are proposed to be provided so that three units could be kept dewatered at one time during the period of construction. A manhole has been provided in the draft tube liner on the downstream side which is approachable from the downstream gallery at elevation 622. This manhole would serve as an access into the draft tube for inspection and repairs.

The intensities of load at various points of the draft tube slab have been determined by assuming dispersion of superimposed loads at 45° in all directions from its point of application. The draft tube top below elevation 622.00 has been designed as a slab to carry the aggregate loads as computed. In the lined portion, the side walls of the draft tube are comparatively more massive and, therefore, the top slab has been designed as fixed to these sides, while in the unlined portion, the side walls, the intermediate pier and the top slab have been taken to constitute a bent and alternatively a box with the bottom slab. The part of the top slab just upstream of the intermediate pier is restrained from deflection by the intermediate pier which fact has been taken into account in the design of the slab. The intermediate pier is more heavily loaded than the end walls and their differential deformations have also been taken into account in the design of the draft tube.

Various alternatives of making the draft tube bottom slab separate from sides were studied but ultimately from certain practical considerations it was decided to make it monolithic with sides, and the bottom slab has been designed accordingly as a composite part of the draft tube.

The bottom of the draft tube at exit end is at El. 608.00 and a slope of 1 in 5 connects it with the normal river bed level which is at Elevation 624.50.

The divide walls in between the adjacent draft tubes will be extended to provide a regulator about 85 ft. downstream of the exit ends of draft tubes with sill at elevation 624.50 and top at El. 638.00. This regulator will ensure a minimum tail water level to prevent cavitation during the early stages of development of load. This regulator will also make it possible to dewater any draft tube in the event of a draft tube gate getting stuck up due to collection of silt etc., in the gate grooves. The regulator will have four additional piers in between the adjacent divide walls so that clear span of each bay of the regulator is 9 feet only. Double grooves will be provided in each pier and divide walls for two sets of planks. A gangway slab will also be provided over the piers. During floods when tail race level is higher than elevation 638.00, the regulator will remain submerged under water.

(b) **Scroll Casing, Speedring and Turbine Pit :**

On account of high head involved, a steel scroll case has been provided which has been designed to be strong enough to take the full water pressure. It is, however, not supposed to support the weight of surrounding concrete and other superimposed loads. The concrete round the spiral has, therefore, been designed independently and the loads coming on the beam round the scroll casing have been calculated in the same way as for the draft tube. Water pressure during floods from the downstream side has also been considered in the design of concrete beam. Concrete round the spiral casing has not been designed to withstand any water pressure and a suitable mastic filler has been placed in between the scroll casing and the concrete so that only a little force, if at all, due to the water pressure inside the scroll case is transmitted to the concrete. The filler will also ensure that dead weight of concrete and machines is not transmitted to the scroll case liner.

The penstocks are of 16 feet internal diameter which is reduced to 15'3 13/16" by a taper piece connecting it to the scroll case. Various tappings for cooling water, pressure gauge and flowmeters are located in this taper piece and a manhole has also been provided in it to give entry into the scroll case and penstocks for inspection and repairs. The scroll case is of welded construction with plate thickness varying from $1\frac{1}{8}$" to $\frac{5}{8}$".

The speed ring weighs over 50 tons and has twelve guide vanes round its circumference. The speed ring and scroll casing will be supported during erection on twelve concrete piers. After the scroll casing has been erected in place, but before connecting it on to the taper piece, it will be subjected to a hydraulic pressure test of 190 lbs./sq.inch.

The turbine pit liner above the speedring consists of $\frac{3}{8}$" thick plate extending upto elevation 650.50. Access to the turbine pit is obtained by means of a stair from the turbine floor at El. 650.00.

(c) Generator Supports :

The generators and turbines will be supported on the concrete around the turbine pit, and no steel supports have been provided. The concrete around the turbine pit is being suitably reinforced to take the short circuit torque of the machine, which is equivalent to a 500 tons peripheral load, as well as the vertical load and impact. The vertical load of the turbine and generator amounts to 560 tons.

(d) Superstructure :

It has been found economical to locate the generator floor 17 ft. below the maximum possible tail water elevation of 680.00. The downstream wall of the power house will thus have to retain water upto this elevation. To expedite construction, it has been decided to provide steel columns for the powerhouse superstructure and for supporting the gantries for 90 ton cranes. The columns, the cross beams and truss have all been designed as welded steel structures so as to economise on steel and cost. If the downstream wall and columns were made monolithic, the columns would have to bear the water pressure from the tail race and would have been very heavy and uneconomical. The downstream wall upto El. 683.00 has, therefore, been separated from the columns and has been designed as a cantilever from the concrete below elevation 650.00. Suitable water stops have been provided in the joints between the downstream wall and columns to prevent entry of water from the tail race into the powerhouse. The columns have their foundations at elevation 650.00 except one column coming over penstock opening which starts at elevation 663.00. Each unit bay has four columns on the upstream and four columns on the downstream side, connected together across the powerhouse at their tops by roof trusses. The roof trusses are pin jointed to the columns. Longitudinal cross beams connect the columns at elevations 684.00, 702.42 and 718.06 both on the upstream and the downstream sides. A column on the upstream with corresponding column on the downstream, together with the roof truss constitute a transverse frame. In the longitudinal direction the four columns of one side together with the three cross beams constitute a rigid frame work. The design of columns and beams has been worked out accordingly. In the erection bay the steel columns have their foundations at elevation 683.00 and, therefore, the cross beam at elevation 684 has been omitted.

The walls above elevation 683.00 have been designed as panel walls supported from column beam frame work. The whole superstructure has been designed to be safe for seismic acceleration of g/10.

The gantry girder is also a welded steel plate girder and carries the crane rail for the two 90 ton electric overhead travelling cranes. Apart from the main hook each crane has an auxiliary hook of 25 tons lifting capacity. Both the cranes are provided with limiting switches and thrust operated brakes both for cross travel as well as longitudinal travel to prevent the trolly or the crane from striking at ends. Accesses to the cranes are provided from the two end walls of the powerhouse.

(e) Roof :

The powerhouse roof has been supported on welded steel trusses. The trusses carry the purlins over which precast reinforced concrete slabs have been laid. The roof will be rendered water-proof by laying a 3 ply membrane of Tarfelt finished with gravel on top. The roof slopes from the centre towards the upstream and the downstream sides and downpipes have been provided for rain water in the upstream and downstream walls.

(f) Erection Bay :

The erection bay is situated at the east end of the power house. The main erection floor is at El. 683.00 which is also approximately the general ground level as well as the level of approach road leading to the power house. The erection bay consists of mass concrete upto elevation 650.00 which forms the lowermost floor. The second floor is at elevation 663.00. Reinforced concrete beams and slabs have been used for the floor at elevation 663.00, but due to heavier loads on the main erection floor at elevation 683.00 steel beams and R.C.C. slabs have been used for this floor. Heaviest single piece of equipment would be the rotor which will weigh 165 tons and, therefore, two rotor erection pedestals rising from solid concrete at elevation 650.00 have been provided so that work can be carried on two rotors at a time. The 60 mVA 11/132 k.V. step up transformers which have been located on the deck upstream of the powerhouse will be brought on a trailor up to the erection bay, where they will be unloaded by cranes

and put on the rail tracks provided in the erection bay floor for the purpose. Untanking space for the transformers has been provided in unit No. 1 as sufficient head room is not available in the erection bay.

(g) **Control Blocks :**

Control building is situated at the south-east corner of the erection bay as shown in the plan (fig. 2). The basement floor of the building at El. 673.00 will serve as cable floor and will receive all the incoming cables from the powerhouse through a cable gallery which is located at elevation 650.00 upstream of the powerhouse and then slopes upto elevation 673.00 along the erection bay. A cable tunnel at the north-east corner of the basement floor will carry cables to the switchyard.

The ground floor of the control building which will house the main control equipment is at elevation 684.00. The battery room, the 11 k.V. switchgear room and service board room are also located on this floor. Space has also been provided in the control room for panels which may be required for future developments.

The first floor of the control building which will provide office accommodation is at elevation 699.62. Air-conditioning equipment and carrier communication equipment will also be located on this floor.

The control building with all its floors has been designed as reinforced concrete beam and slab structure, safe for a seismic acceleration of g/10. The roof of the control building has also been provided with water-proof membranes similar to the powerhouse roof. The northwest corner of the control building has a vertical access shaft housing an electric lift and a stair case. The powerhouse floors and galleries are also accessible from this shaft and elevator landings at various floor levels are connected by galleries to the main powerhouse floors.

(h) **Joints and Waterstops :**

As previously stated all the unit bays and the erection bay are separated by means of joints. These joints are made water-tight by a system of water-stops. In joints which are likely to be submerged under water an asphalt seal has also been placed. Tubes will be embedded in this seal through which steam can be passed for heating the asphalt. A formed drain has been provided to collect any leakage, if at all.

Control building is situated on three dam blocks which are separated by contraction joints. These joints continue into the superstructure of the control building and waterstops have been provided to prevent seepage of water into the building.

(i) **Ventilation and Air Conditioning :**

The use to which various spaces will be put, their locations, occupancy and probable dampness were some of the factors kept in view while designing the ventilation system of powerhouse. The generator room will be ventilated by means of seven exhaust fans to be installed in the upstream wall near the roof, one in each bay. The turbine floor and the lower galleries have been provided with forced inlet ventilation so that air there may not become stagnant or damp. Air is forced through ducts into the turbine floor and also into the gallery at elevation 622.00. Air can also be forced into draft tube dewatering gallery when required. The air after circulation will return to the generator floor through the various openings in the floors and then exhausted out by the exhaust fans. The air thus gets fully circulated.

The oil storage and oil purification rooms at El. 650.00 below erection bay are provided with a separate system of ventilation as part of fire protection system. Duct from these rooms is taken separately to atmosphere and an exhaust fan is provided for necessary ventilation.

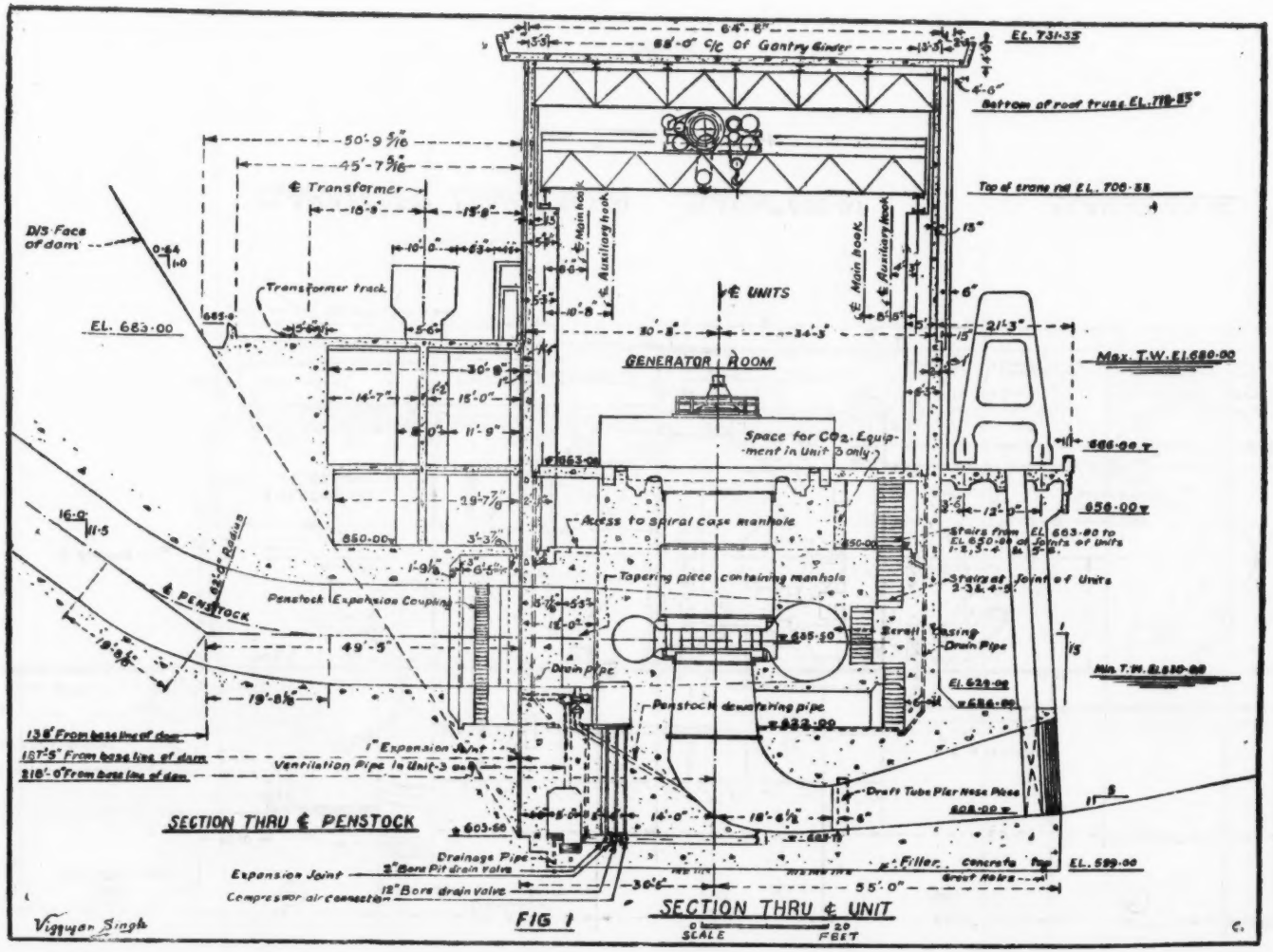
The two upper floors of the control building will be air-conditioned. Sufficient openable windows have also been provided for natural ventilation, if required.

(j) **Sixth Unit :**

As mentioned at the beginning of this article only five units will be installed at present. It was first proposed to erect the draft tube liner for unit number six also but subsequently it was decided that to allow greater flexibility for the future suppliers of this machine, only the downstream unlined part of the draft tube be constructed. Thus a full blockout upto rock level was left for the draft tube liner, speed ring, scroll casing and the machine itself. This makes the design of this unit below elevation 650.00 different from the other units. The concrete on downstream side of the blockout will have to retain water upto elevation 680.00 during maximum flood and the structural design of sixth unit had to suit this condition.

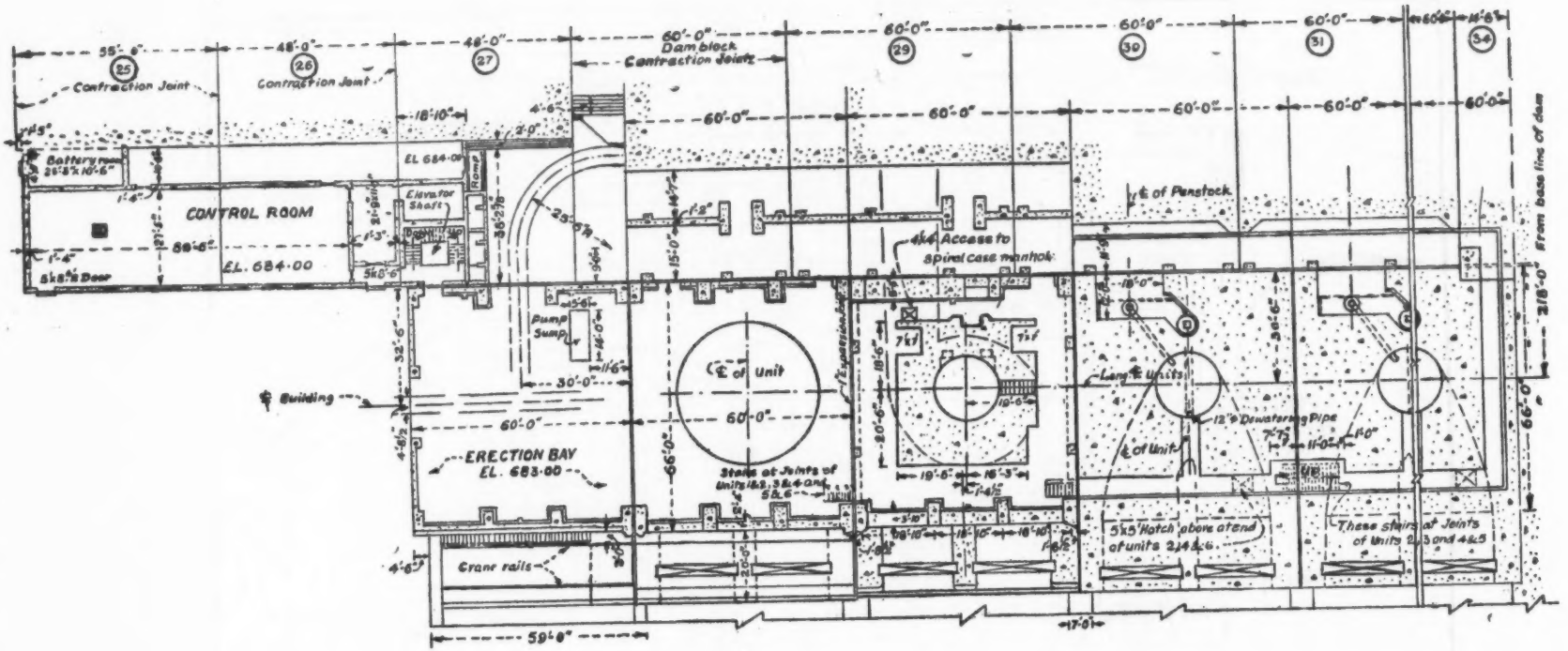
(k) **Architectural Treatment :**

While utility and economy have been the governing features of the powerhouse design, the architectural aspect is being kept in view for fixing details. Due consideration is being given to the appearance of the powerhouse and control building. The architecture is proposed to be simple and forthright and to fit in with the surroundings. The powerhouse is situated at the toe of dam blocks. For similarity of appearance no special finish is proposed to be given on the outside surfaces of the powerhouse and control building. High dam being situated very near on the upstream side, the main faces of the power house and control building, from appearance point of view, are the downstream and the east faces. The length of the powerhouse building is considerably more than its height, further the building is dwarfed by the dam



Vigyan Singh

FIG 1



PLAN AT 683·00 PLAN AT 663·00 PLAN AT 650·00 PLAN AT 622·00

Scale:- 0 32 Feet

FIG. 2.

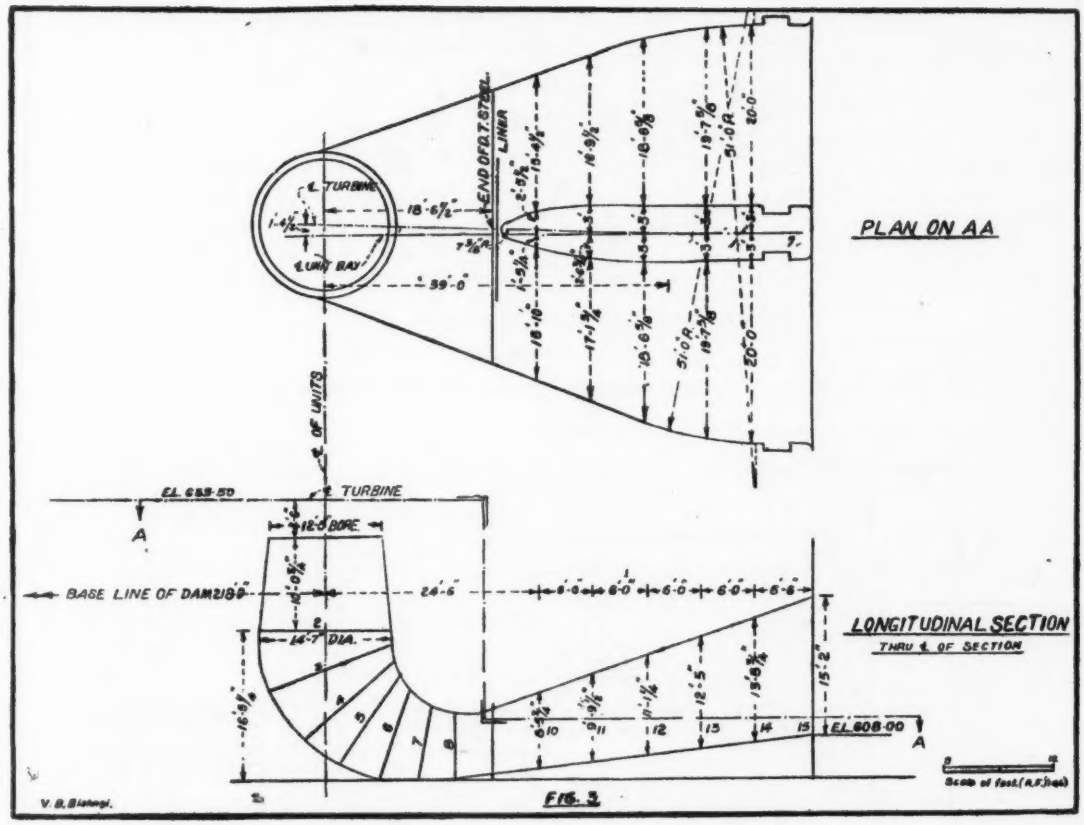
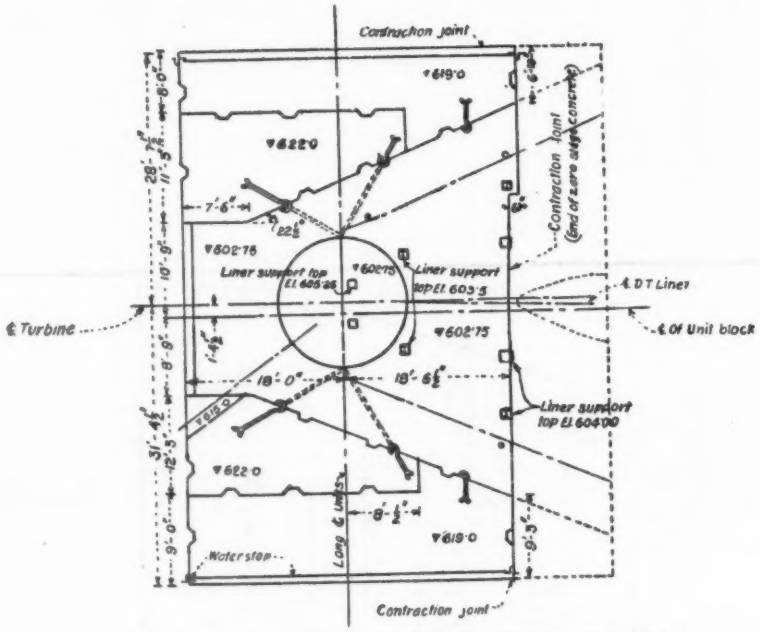


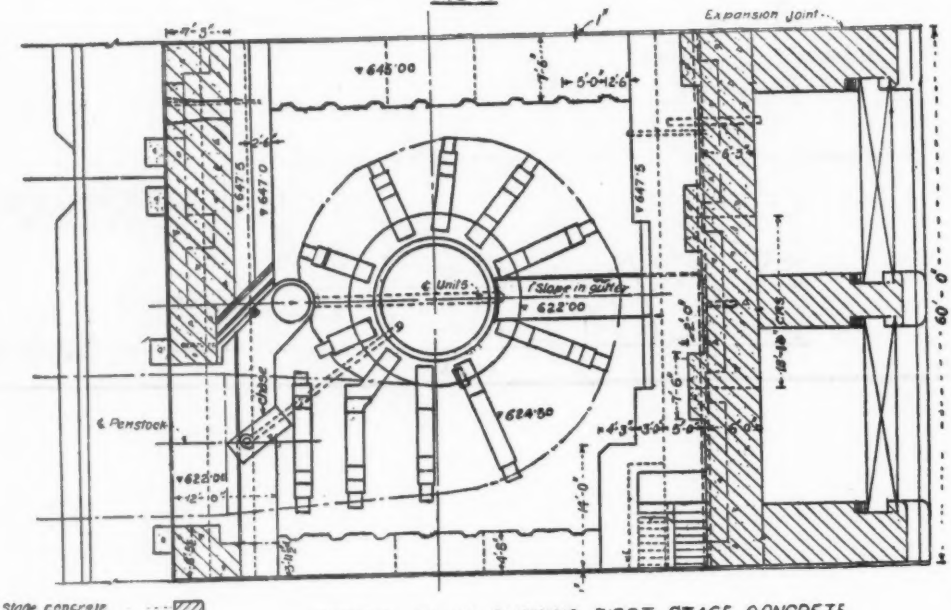
FIG. 3

V. B. Bhatnagar.



TYPICAL PLAN SHOWING ZERO STAGE CONCRETE

FIG. 4



TYPICAL PLAN SHOWING FIRST STAGE CONCRETE

FIG. 5

in the background hence it was considered desirable to give a vertical treatment on the downstream face. The superstructure columns at the unit joints have been suitably projected out to give the desired effect. Similar projections though not so marked are provided on the downstream face of control building.

The internal finishes also follow the same requirements of utility and economy. All galleries below elevation 650.00 in powerhouse, the floors at elevation 650.00 and 663.00 under erection bay and the basement floor of control building are finished in concrete. The turbine and generator floors of power house and top floor of erection bay are proposed to be finished with coloured cement. The main floors of control building are proposed to be finished in mosaic, and a false ceiling will also be provided. The space above the false ceiling will be utilised for air conditioning ducts.

STAGES OF CONSTRUCTION

(a) **Zero Stage Concrete :**

This consisted of certain preliminary concreting which may act as anchoring and supporting concrete for the draft tube liner. Fig. 4 gives a plan of the zero stage concrete. The draft tube liner rests on supports raised from the bottom and is also held in position by anchors from the bottom slab and sides. This facilitated accurate positioning and levelling of the liner.

(b) **First stage Concrete :**

After erection of the draft tube liner in place the first stage concrete was poured. Figure 5 shows the first stage concrete in place. As seen from it this consisted of the completion of draft tube, raising the concrete at sides to elevation varying from 645.00 to 647.50 according to requirement and completing the superstructure. A blockout was left for the speed ring and the scroll casing. With this arrangement the powerhouse cranes were available for the erection of speed ring and scroll casing, thus expediting and facilitating the work. Concrete piers were constructed for supporting the speed ring and scroll casing till second stage concrete was poured.

The first stage concrete was checked and suitably designed for all loads and forces likely to come over it during the construction period when the speed ring and scroll casing are being erected.

(c) **Second stage Concrete :**

This consists of the concrete around the speed ring, scroll casing and all other remaining powerhouse concrete right upto the generator floor level.

OTHER CONSTRUCTION DETAILS

Work on powerhouse concreting was started after the monsoons of 1957 when zero stage concrete for the five bays was laid. The draft tube liners were then erected. To save time the draft tube liners, each weighing about 13 tons, were assembled by means of a mobile crane in an assembly area outside during the period that the zero stage concrete was being laid. Assembled liners were then lifted by means of the cableway and put on the previously prepared foundations. This saved a lot of erection time. The next stage was to embed the liners, and lay the first stage concrete as described above as also to complete the erection bay. After completing this stage speed ring and scroll casing were erected, and second stage concrete is being laid at present. This is expected to be completed shortly for the first unit and erection of machines will then be started.

To work upto the above tight schedule it was necessary to carry on the work on the powerhouse throughout the year i.e. even during the monsoons when the river was in flood. To achieve this a coffer dam with top at elevation 683.00 was constructed to enclose the powerhouse area prior to monsoons of 1958. The intake blocks of the dam were also raised sufficiently to act as a coffer dam from the upstream side. This enabled the work to be carried on during the monsoons.

(Continued from page 93)

POWER LINE CARRIER COMMUNICATION SYSTEM FOR RIHAND GRID

band pass directional filters no. 4 and 5. When Faizabad is called the H.F. band finds a direct path to the Sohwal-Faizabad line section through the directional filters.

The adopted double side band carrier telephone equipment requires two frequency bands of 5 kc/s width each for the go and return channels. Thus 15 pairs of carrier frequencies are available for double side band carrier telephone equipment within the range of 150-300 kc/s which will be ample for future extension.

Although some overall economy could be affected by the use of super-imposed channels technique for telemetering purposes yet it has not been favoured on account of its restricted extension facility. In the case of telemetering system employed there is great scope for addition of more channels.

Rotary convertors operated from 110V or 220V station batteries have been provided to serve in the case of mains failure. The capacity of the batteries will be enough to supply the rotary convertors for two to three hours. Switching over from mains to rotary convertors will be automatic. The short starting period of rotary convertors is bridged over by means of condenser batteries.

As mentioned earlier the protection equipment in the fixed frequency districts works in conjunction with the carrier telephone cabinets in the telephone band itself, therefore, in case of short operating time of in-band frequency shift carrier protection equipment the telephone circuits will be temporarily switched off. Normally the operation time will be less than 50 to 100 milli-seconds.

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Sg 711/4

Hydraulic Turbines for Rihand Power Station

By R. V. MATHEWS

THE power station, situated at the foot of the dam on the Rihand River, has been arranged to accommodate six hydro-electric sets. The present development includes the installation of five sets. The alternators and the turbines of each set are designed and manufactured by the English Electric Co. Ltd. Installation of the sixth machine will be the subject of a later development.

The turbines, which are of the vertical reaction type run at 150 R.P.M. and each develops 70,000 H.P. at 199 ft. head and up to 77,000 B.H.P. at heads above 224 ft. The design of the machine generally follows the English Electric Company's established practice for a medium head Francis turbine and includes a number of interesting features. Fig. 1 shows the general arrangement of the machine. It will be noticed that extensive use has been made of electric welding, all the main structural members being of fabricated construction. Many of the major components of the machine were model tested in the Company's laboratories and in particular the runner was subjected to extensive model tests.

We propose to give a short description of some of the major components and briefly describe the tests carried out on them.

RUNNERS

Model Tests

Tests were carried out in the Company's hydraulic laboratories at Rugby to investigate the complete

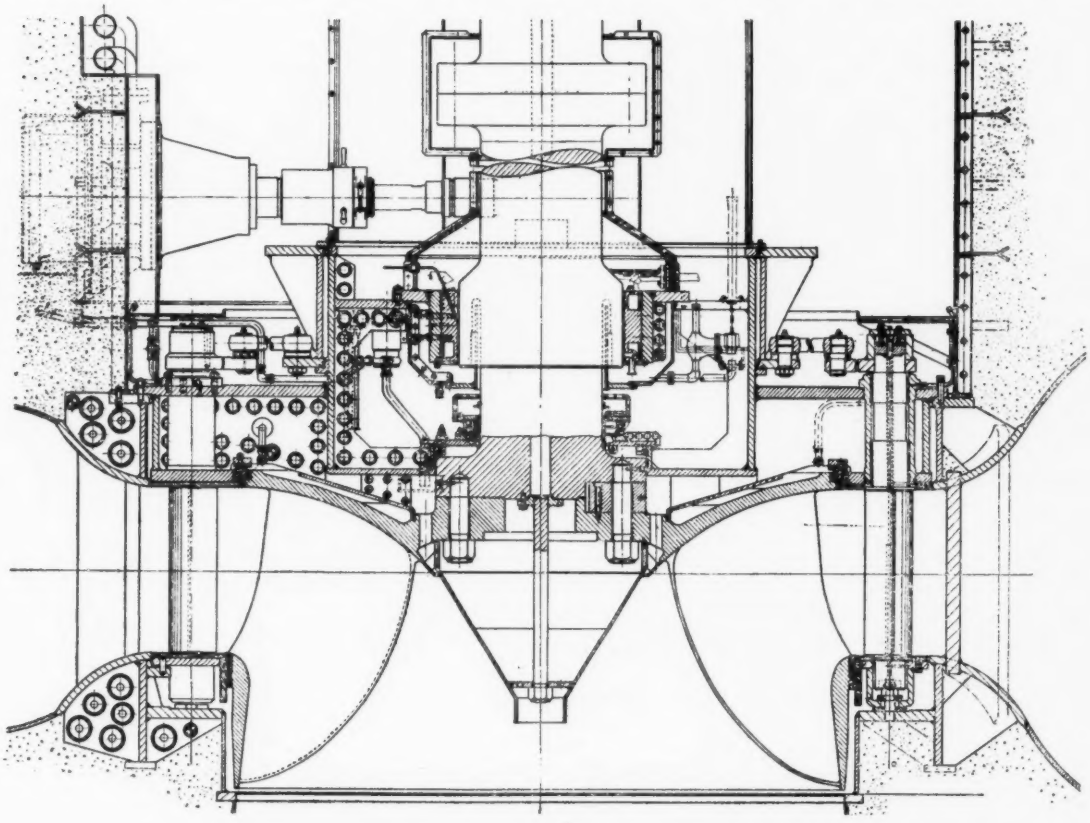

Fig. 1—Sectional Assembly of Turbine.

range of operation of the full size machines. A total of eight test berths are available and two of these were used for the Rihand runners.

The design of the runners followed the usual procedure. A model is made from the design drawings and tests are carried out to check its performance. During these performance tests the runner is set relative to the tailrace level in such a way that no chance of cavitation in the runner passage exists.

Modifications to the runners are then made and the model re-tested until the performance meets the specified requirements.

Once a satisfactory model has been achieved, it is put in the cavitation test berth where absolute pressure in runner passages can be reduced to check that over the complete operation range no danger from cavitation exists. Sometimes modifications to the blade form are necessary to achieve this. These modifications are then made and if at all extensive the model is again checked for performance. Experience gained from previous tests is of course of the greatest help in reducing this trial and error procedure to a minimum.

(a) *Performance tests.* The test berth used is shown in fig. 2. The water is re-circulated, being pumped

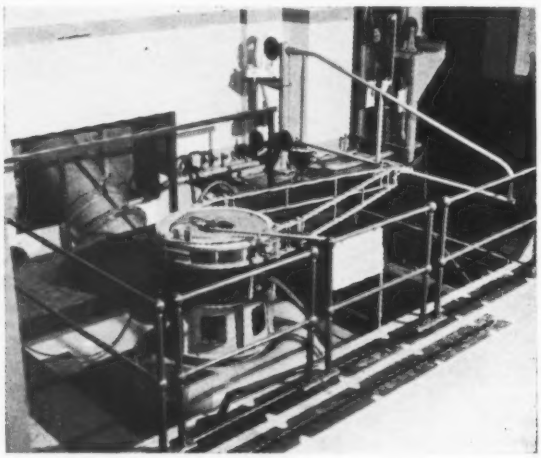

Fig. 2—Performance Test Berth in the Hydraulic Laboratory at Rugby.

from a sump tank, passed through the model and returned to the sump tank after passing over a weir in the tailrace. A 22″ dia. model runner was used and this developed about 25 H.P. under the 14 ft. test head.

The discharge from the model is carried through a long rectangular tailrace channel fitted with screens to eliminate turbulence and led to a standard rectangular weir to measure the discharge. The head of water above the two metre wide weir crest is measured by a float which operates a scale whose magnified image is projected on to a screen for easy and accurate reading. By using these readings and the standard weir coefficients the discharge can be computed.

The head on the machine is transmitted from the measuring point using two transducers relaying an air pressure exactly equal to the submergence of two probes situated in open chambers. One chamber is connected to a ring of static tappings on the penstock and the other to a transverse pipe across the tailrace channel. The air pressure lines are connected to a differential water manometer fitted with a float operated moving scale. An optical device projects the magnified image of the scale on to a screen for easy and accurate reading.

The turbine speed is measured by an electronic counter, triggered every 1/10 revolution and timed by a crystal maintained oscillator operating a synchronous clock with one minute contacts.

The turbine torque is measured by an automatic Prony dynamometer which absorbs the turbine output. This Prony brake operates on a 3ft. brake drum and is clearly seen in fig. 2. Friction due to the weight of the dynamometer swinging arm is eliminated by supporting it on air lubricated pads. The length of this arm is checked periodically to allow for brake shoe wear. The load on the end of the arm is provided by standard weights and a vernier Avery scale.

Individual tests are carried out over a test period of 1 minute and three minutes are allowed between tests to ensure steady conditions. These precautions and the advanced apparatus available in the test berths, lead to a very high standard of accuracy for the tests.

From these results, the efficiency and output of the prototype for a large range of heads can be computed and the guarantees for runaway speed can also be checked.

(b) *Cavitation Tests.* The cavitation plant used (see fig. 3) is of the closed circuit type. The pump at the base of the plant pumps water through four large resorber tanks connected in series and thence through a 36″ venturi meter to a large tank which holds the turbine spiral casing. The purpose of the resorbers is to put back into solution the air previously removed due to the low absolute pressure. Water discharges through the turbine and draft tube back to the pump inlet for re-circulation. Absolute pressure in the plant is altered by regulating the datum pressure whilst maintaining constant the head across the turbine.

For Rihand the runner model was 22″ dia. and was tested under 35 ft. head.

The discharge through the turbine is measured by the calibrated venturi meter, the differential head across the meter being measured by a mercury manometer.

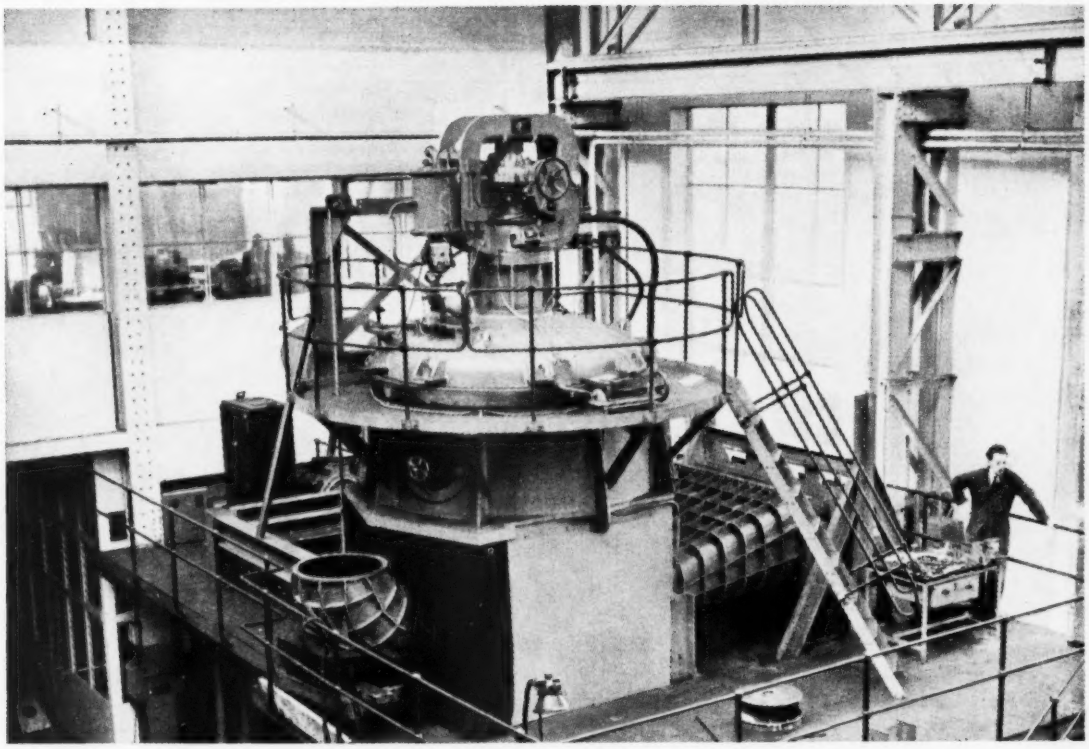

Fig. 3—Cavitation Test Berth in the Hydraulic Laboratory at Rugby.

Head across the turbine is measured with a differential mercury manometer connected to the spiral case inlet and the draft tube exit. The turbine speed is measured electronically as described under performance tests.

The turbine torque is measured and the output absorbed by a Heenan and Froude dynamometer arranged so that turbine bearing and seal losses are transmitted to the dynamometer.

The cavitation tests were carried out at the operating points found during the performance tests, to determine what limitations were imposed by the tailwater elevation. The absolute pressure was gradually reduced keeping the head on the machine constant thus decreasing the cavitation factor. By plotting the power output against cavitation factor the critical factor was found. This critical cavitation factor was compared with the factor found from the established setting of the plant and the safety margin was determined. At the same time visual inspection of the flow in the machine could be made using a stroboscope through the perspex draft tube cone. This enabled the parts of the runner most likely to be affected by cavitation to be found.

Construction

Once the model tests had been completed production of the cast steel prototype runner was started. The drawings of the model runner were carefully scaled up to make the prototype drawings. These were then sent to the founders so that the necessary patterns and core boxes could be made.

During the cavitation tests the areas most likely to be attacked by cavitation were found to be the skirt of the runner and the backs of the blades adjacent to the skirt. When the runner was cast recesses were left over these areas and they were later filled with stainless steel strips welded to place. When the runner was ground these areas were blended with the mild steel part of the runner to give a smooth streamlined surface. The passage widths between blades at exit from the runner were measured and adjusted by chipping and grinding to template to the dimensions determined from the model runner.

The runner is provided with high carbon steel labyrinth sealing rings on the rim and the crown. Leakage water passing through the seal on the crown is evacuated into the draft tube through a series of cored holes in the crown of the runner thus relieving the axial thrust on the runner. The water passes to the cored holes through a passage divided by radial ribs. These ribs prevent the formation of a vortex and so the inward radial flow is not impeded by centrifugal effects.

SPIRAL CASING AND SPEED RING

Owing to the large physical size of the machines it was necessary to split the speed ring into four parts in order to pass it through the railway loading gauge. The photograph in fig. 4, taken in the works during manufacture, shows quarters of the speed ring in the

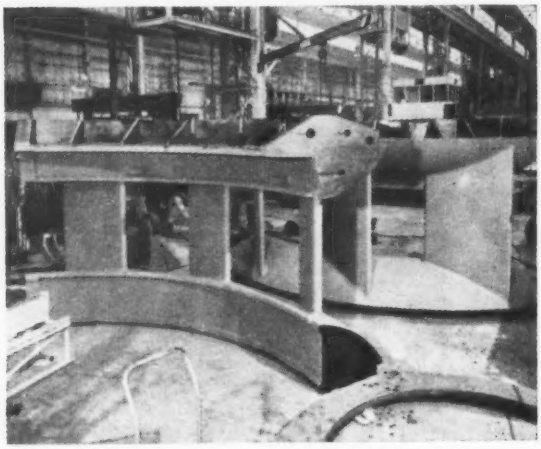

Fig. 4—Quarter Segments of Speed Ring.

inverted position. The joint faces can be clearly seen. The holes in the flanges are only pilot holes for use during manufacture and the final holes have yet to be drilled. Fig. 5 shows the speed ring assembled ready for erection of the spiral casing plates. The joints were made using $3\frac{1}{2}''$ and 4″ dia. high tensile bolts and owing to the large forces involved these were heat tightened. Each bolt was drilled $1\frac{1}{4}''$ dia. throughout its length and electric heaters were inserted into the bore. The bolt was heated about 200°F above the ambient temperature and the nut rotated a calculated amount depending on the pitch of the thread and bolt length. This produced a certain prestress in the bolt which could be checked by measuring the initial and final lengths of the bolt and computing the bolt stretch. By this means it was possible to tighten the bolts to a larger stress than would be possible by slogging and take full advantage of the high tensile material.The stay vanes can also be seen in the photograph. They are of 3″ thick plate and were designed to carry the full water load when the spiral casing is full and the weight of the generator and superimposed concrete when the spiral casing is empty. To complete the assembly of the speed ring at site a seal weld was made round the periphery of the joint.

Size considerations also necessitated the spiral case being sent to site plate small. The spiral plates were rolled and trimmed and assembled on to the speed ring in the works. All the plates were then match marked to facilitate the final assembly at site. After completion of the welding of the spiral plates at site the casing was subjected to a water pressure test to ensure that all welding was perfectly sound.

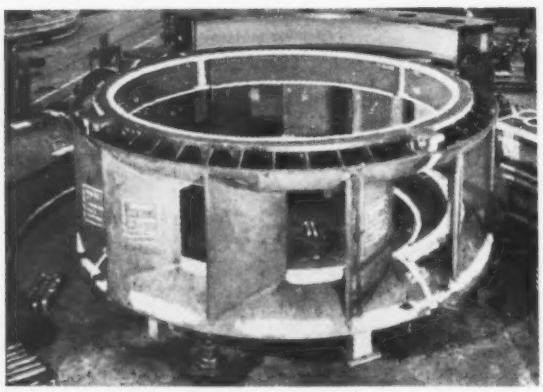

Fig. 5—Speed Ring assembled in Works.

DRAFT TUBE

As in the case of the spiral casings these were so large that they had to be sent to site plate small and welded together there. Fig. 6 shows the draft tube match marked and assembled at works. The photograph gives a good idea of the size of the draft tubes. At site grouting irons were welded to the ribs and to the plate between the ribs. This ensures that the draft tube is firmly bonded to the concrete and prevents vibration of the steel plates.

HEAD COVER

For a number of years tests on head covers for water turbines have been carried out at the Company's Mechanical Engineering Laboratories at

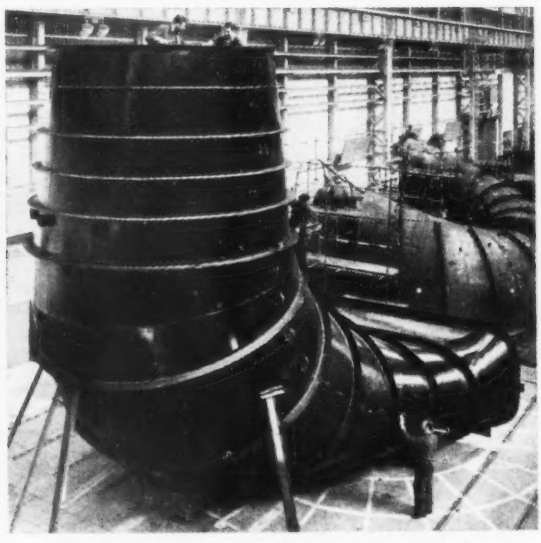

Fig. 6—Trial Assembly of Draft Tube in Works.

Whetstone. A whole range of covers for turbines of different specific speeds has been tested and the stresses and deflections of the covers have been measured. These test results have confirmed a theoretical analysis so that the stress and deflection for a new cover can now be estimated. The formulae consists of a number of simultaneous equations which have to be solved and it has been found convenient to make a programme so that the calculations can be made on a DEUCE computer. This procedure was used for the Rihand machines. A photograph of the head cover assembled on the works is shown in fig. 7. It is in the inverted position but shows the massive top and bottom decks.

A full size mock-up in ply wood of part of the top cover, shaft, bearing housing and gland was also made. This enabled a check to be made that adequate access to the main shaft gland had been provided. Again owing to transport limitations, the head cover was sent to site split in halves. Heavy joint flanges and bolts were provided to facilitate assembly at site.

GUIDE VANES

Fig. 8 shows the guide vanes during manufacture. They are of fabricated construction. The blade of the vane was made by knifing a plate in a die to obtain the correct profile. The main trunnion and pivot trunnion were then welded to the blade and the whole was then stress relieved before final machining.

Since no main valve was to be installed at the inlet to the machine, it was specified that arrangements should be made to reduce the leakage through the closed guide vanes to a minimum. This has been accomplished by providing rubber seals at the junction between the guide vanes and the pivot ring and top cover, as well as along the vertical sealing face between guide vanes. These seals consist of rubber strips bonded on to steel backing strips. The seals at top and bottom of the vane were inserted in slots cut in the face of the pivot ring and top cover and the seal in the guide vane vertical face was inserted in a slot cut in the nose of the vane. This slot at the nose of the vane can be clearly seen in the photograph.

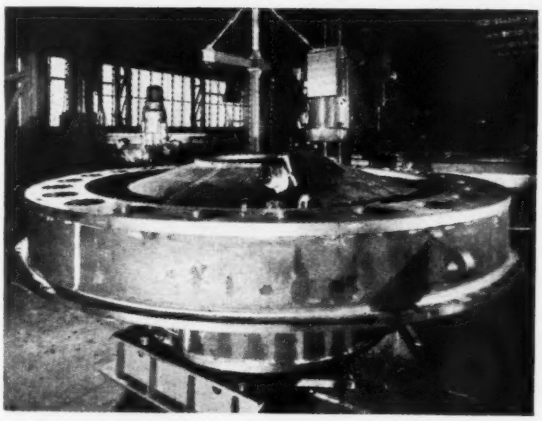

Fig. 7—Head cover jointed and partly machined.

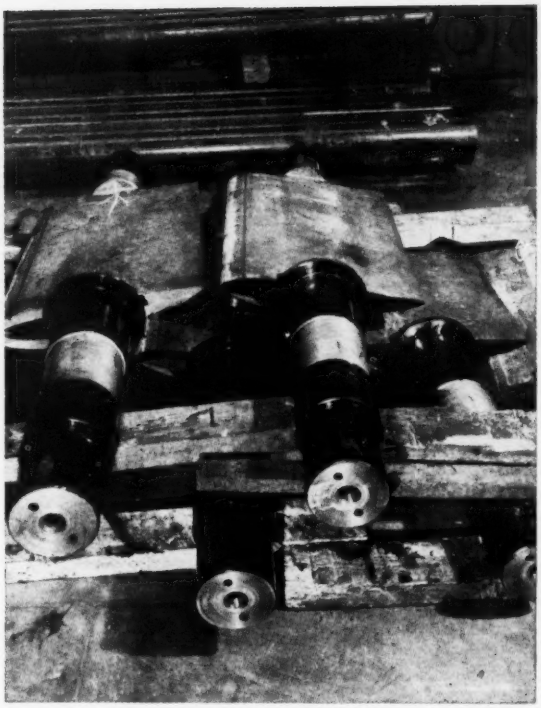

Fig. 8—Partly finished Guide Vanes ready for final machining operations.

Special rubber seals are used for sealing the guide vane trunnions where they pass through the top cover. These seals bear on the stainless steel guide vane collars and have been the subject of extensive tests in the Company's mechanical laboratories at Whetstone to determine the interference between the rubber and sealing face necessary to ensure perfect sealing with minimum wear. Special care was taken to provide a smooth finish on the sealing face of these collars to reduce the wear. The collars can also be seen in the photograph.

It will also be noticed that a hole has been bored in the centre of the main trunnion. A pipe was later inserted in this hole to feed grease to the pivot trunnion bearing. A grease connection was also taken to the annulus formed between the pipe and the bored hole. This feeds grease to the main trunnion bearing.

MAIN BEARING

This bearing is of the English Electric self lubricating type. It was developed and tested in our laboratory some years ago and is now in successful operation in many parts of the world. The bearing consists of six separate pads equally spaced round

the shaft with a small gap between each pad. These gaps are kept filled with oil by the pumping action of a circumferential groove cut in the bearing surface of each pad. The oil is drawn into the groove from a lower reservoir. Oil from these gaps is drawn in under the pads and keeps them lubricated. Surplus oil fills an upper reservoir and returns to the lower reservoir via stand pipes which act as spillways and maintain the level in the upper reservoir. To cool the bearings, the pads are made hollow, connected together in series and have water circulated through them. This has the advantage that the cooling water is in close contact with the bearing surface where the heat is developed. The cooling water pipes connecting the pads are arranged to be below the oil level in the upper reservoir in order that no atmospheric moisture can condense on them and contaminate the oil.

Experience has shown that very little water circulation is necessary to cool the bearing and that the separate pads make any adjustment of the bearing clearance comparatively simple.

The installation of a self-lubricating bearing makes it unnecessary to provide a lubricating pump set, together with a standby set and the associated control equipment and flow relays. It also eliminate the external cooler, and hence considerably simplifies erection and operation.

MAIN SHAFT GLAND

This has been the subject of extensive development in the Company's mechanical laboratories. The seal consists of pairs of carbon rings, three pairs being fitted in each gland, and each pair forming a sealing member. Of each pair of rings, one ring consists of series of segments with wedges between them. The other ring arranged axially above the first ring consists of segments which cover the wedges in the first ring. These rings are held by garter springs against a stainless steel sleeve fixed on the shaft and held axially against a static stainless steel sealing face in the gland box by the force of one of the garter springs acting on an inclined face. Two pairs of carbon rings are arranged to prevent leakage of water out of the machine and the third set prevent leakage into the machine.

Cooling of the gland is achieved by passing clean cooling water through the gland box. In the event of the water which operates the turbines containing suspended silt, the pressure of the clean cooling water in the gland box can be maintained at a higher pressure than the water in the turbine thus ensuring that no contaminated water enters the gland box.

The stainless steel sleeve is applied to the shaft in halves which are then welded together. The contraction of the longitudinal welds is adequate to hold the sleeve rigidly on the shaft.

SHAFT

The main shaft which is a carbon steel forging, is shown being machined in fig. 9. The collar on the

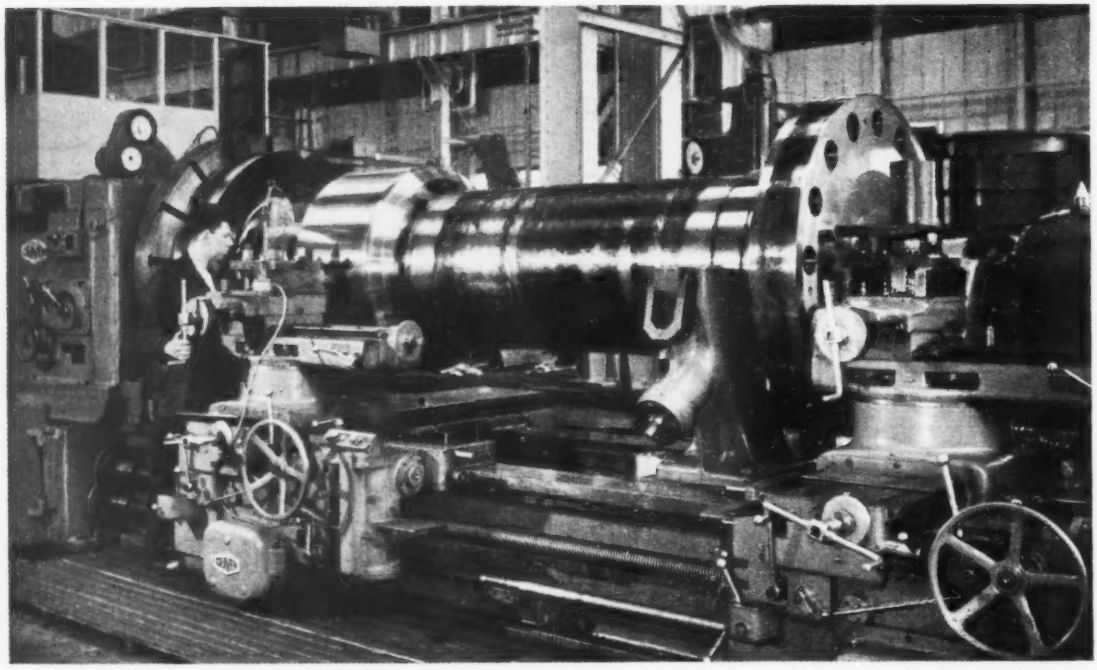

Fig. 9—Main Shaft being machined.

shaft is undercut to receive the inner lip of the lower bearing oil reservoir. The outer diameter of the collar is the bearing surface. Provision has been made below the collar for the stainless steel sleeve for the gland.

GOVERNOR

The governor consists of five main parts:
(a) The governor alternator.
(b) The actuator.
(c) The pilot servomotor.
(d) The main distributing valve.
(e) The guide vane servomotors.

The governor alternator is directly driven from the top of the main shaft. It is connected to the motor which drives the pendulum or speed sensitive device in the actuator. Also in the actuator is a small distributing valve which is operated by the pendulum. This actuator distributing valve supplies oil to the eight inch dia. pilot servomotor. The actuator is placed on top of the pilot servomotor and hence can supply it with oil without the need of pipe connections. This actuator-pilot servomotor assembly is in the governor cabinet which is combined with the instrument panel and is situated on the alternator floor. The main distributing valve which is immediately below the pilot servomotor on the turbine floor is operated by a swinging arm. One end of this arm is connected to the pilot servomotor through a vertical operating shaft and the other end of the arm is connected by a cable to the main guide vane servomotors, (return motion connection). The main distributing valve supplies oil to the guide vane servomotors and these operate the guide vanes by way of the regulating ring. It will be noticed that the governor is merely a servo mechanism, which uses the speed signal from the main shaft and converts it into large forces for operation of the guide vanes. To avoid any chance of hunting of this mechanism precautions are taken to ensure that all links and pins are loaded constantly in one direction so that any back lash due to clearance or wear is eliminated.

The oil supply to the governor is provided by the governor oil pumping set situated on the turbine floor between the units. It consists of two Imo pumps mounted vertically on a sump tank and these deliver the oil at 300 lbs/sq.in. into an air loaded receiver. This receiver acts as a reservoir against sudden governor demands. Should the oil pressure fall due to any cause a pressure relay is arranged to shut the machine down at a time when adequate oil still remains to supply the requirements of the governor for one closing stroke.

The pumping set for the alternator lubricating oil is of similar design and situated in the station adjacent to the governor pump set. This leads to neat appearance and ease of operation of the station.

CONTROL, OPERATION AND PROTECTION OF TURBINE

Normal operation such as starting and stopping the plant are under local manual control at the governor control panel on the alternator floor. The control scheme is generally as for a fully attended station, but full protective features have been provided as for an unattended station. Guide vane stroke limitation. can be achieved manually by limiters on the guide vane servomotors. Stroke limitation is also provided in the actuator and this can be operated by electrical remote control as well as from the governor panel. Provision is also made for the remote indication of the position of this limiter and for indication of the guide vane opening.

SCALE MODEL TURBINE

A model of the turbine and alternator made to scale of $\frac{3}{4}$" to one foot is also being supplied by the English Electric Co. Ltd. This model is power driven, provision being made for operation of the governor gear and rotation of runner and rotor. A photograph of this model is shown in fig. 10 and many of the main features of the turbine mentioned above can be seen.

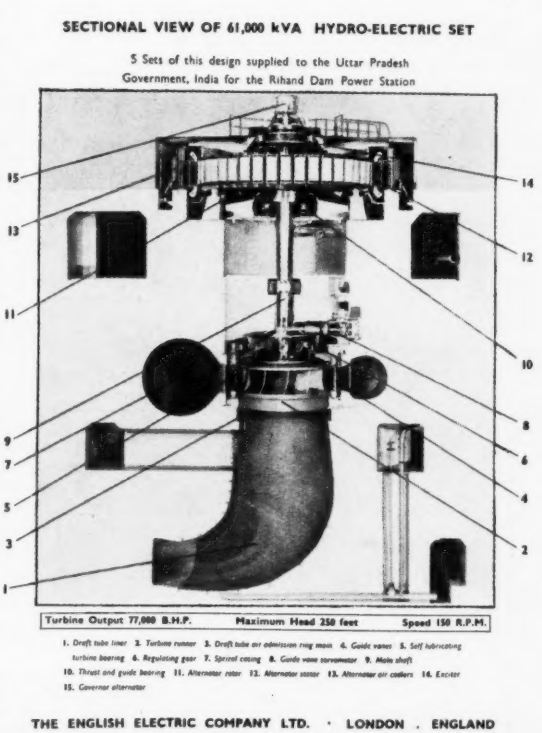
SECTIONAL VIEW OF 61,000 kVA HYDRO-ELECTRIC SET
5 Sets of this design supplied to the Uttar Pradesh Government, India for the Rihand Dam Power Station
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
Turbine Output 77,000 B.H.P.
Maximum Head 250 feet
Speed 150 R.P.M.
1. Draft tube liner 2. Turbine runner 3. Draft tube air admission ring main 4. Guide vanes 5. Self lubricating turbine bearing 6. Regulating gear 7. Spiral casing 8. Guide vane servomotor 9. Main shaft
10. Thrust and guide bearing 11. Alternator rotor 12. Alternator stator 13. Alternator air coolers 14. Exciter
15. Governor alternator
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Fig. 10.

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Outdoor Switchyard at Rihand Power Station

By P. A. KRISHNASWAMY

THE design of an outdoor switchyard of a Hydro-electric Power Station depends upon many factors of which the following are the most important:—

(*a*) The location, nature and dimensions of the site available for the Switchyard.
(*b*) The place occupied by the Power Station in the general scheme of transmission.
(*c*) The number and capacity of the incoming and outgoing feeders.
(*d*) The voltages at which power is generated and transmitted.
(*e*) The ratings and the types of circuit breakers, isolators and other components.
(*f*) The flexibility of operation desired.
(*g*) The need for future expansions or alternations if any.

In order to enable one to understand the above factors with reference to the Rihand Power Station and its outdoor switchyard a brief description of the Power Station and other conditions peculiar to the site of the switchyard are given in the following paragraphs.

GENERAL DESCRIPTION

Rihand Power Station, is located on the downstream side of Rihand Dam at Pipri, a village in the South-eastern corner of Mirzapur district. Six units, each of 55,500 kVA capacity at 0.9 p.f., five initially and the sixth at a later date, are proposed to be installed in the Power Station.

The Power Station will feed a network of 132 kV and 66 kV lines distributed in the Eastern Districts of Uttar Pradesh.

Power is generated at 11 kV and stepped up to 132 kV by means of 60 MVA 3 phase transformers located between the down stream face of the Dam and the Power House building. The conventional unit scheme of generator transformer connection is adopted.

The outdoor switchyard will have five incoming 132 kV connections (ultimately six when the sixth unit is installed). Six outgoing 132 kV line connections, two each to Mirzapur, Moghal-Sarai via Robertsganj and Sone Nagar in Bihar are to be provided in layout. Space is to be provided for three more 132 kV double circuit line connection for future interconnections.

A 66 kV bus arrangement with provision for two incoming connections from the local 132/66 kV 10 MVA transformers, two outgoing 66 kV line connections and two feeder circuits to two 66/11 kV 3 MVA transformers located in the switchyard have to be provided. The outgoing 66 kV lines are intended to feed industrial loads, which may come up in the area. The 66/11 kV, 3 MVA transformers are intended for supplying 11 kV local distribution feeders and also certain non-essential auxiliaries of the Power Station and the Dam through a further transformation of 11/.4 kV.

There are two sites where outdoor sub-station could be laid:—

(*a*) Between Dam and Power House on the upstream side.
(*b*) On the eastern bank, neighbouring Control Room.

In the former place the sub-station would be a high level sub-station with various decks, while in the latter it could be a combined high and low level sub-station similar to those constructed in other parts of State.

The former suffers from the drawback that more than six feeders cannot be taken out.

With the above in view as also the ease in maintenance and inspection on a combined high and low level construction the latter site has been favoured. From economic considerations there is not much to choose as the increased quantity of steel for high level substation is offset against the increase in length of multicore control cables in the latter case.

The space available for sub-station on eastern bank is 131m $\times$ 86m approximately.

With the main power transformers located between the Dam and the Power House, two alternatives were possible for connections between the transformers and the circuit breakers located in the switchyard.

1. 132 kV Oil-filled cables laid in cable trenches or ducts.
2. 132 kV line connections carried over the Power House Building with suitable anchorings on either side.

The second alternative is adopted for Rihand as it is very much cheaper, extremely reliable and requires practically no attendance once it has been installed.

CHOICE OF BUS BAR SCHEME

Double Bus bar scheme has been adopted for the 132 kV switchyard for the following reasons:—

(i) It permits interconnection of one or more generators of the Power Station to a test section without disturbing the main network.

(ii) It permits greater flexibility in maintenance and operation of the switchgear.

(iii) It permits synchronising of a particular Station or generators of a network section independently of the main network.

One of the units of the Rihand Power Station will normally be stand-by and the capacity of the double circuit 132 kV outgoing lines are such that one of the two circuit breakers of the D/C lines can be switched off without any disturbance to the continuity of power supply. For this reason by-pass isolator is not provided for the circuit breaker. Also the isolator between the step-up transformer and the circuit breaker of the incoming line connections has not been provided in accordance with the latest practice.

Single bus bar scheme is provided for the 66 kV switchyard.

CHOICE OF TYPE OF LAYOUT 'LOW' OR 'HIGH'

There are two principal types of layouts in vogue.

(i) With the bus bars and the isolators carried on steel structures and the circuit breakers on the ground on suitable foundations. This type of arrangement is called the 'high type'.

(ii) With the bus bars and the switchgear including the isolators on comparatively low level. This arrangement is called the 'low' type.

The 'low' type of arrangement naturally occupies more ground space than the 'high' type, but is more economical in the structures and gives an unrestricted view of all the switchgear equipments. It is therefore widely used where there are no restrictions in space.

ALTERNATIVE LAYOUTS FOR COMPARATIVE STUDY

The following three alternatives were prepared.

Alternative Layout I.

A 'low' type of layout with 'keel' or one-after the other arrangement of isolators and with 'strung' buses, one of the buses being in the form of a ring around the other in order to accomodate in the same bay width the outgoing 132 kV line connections opposite to the incoming 132 kV line connections, which are terminated on a one level gantry at the switchyard.

Alternative Layout II.

A 'high' type of arrangement with the conventional 'side-by-side' placing of isolators and with strung type of buses. One of the buses in the form of a ring around the other. The outgoing connections are opposite the incoming ones as in alternative I. The overhead connections from the transformers are however terminated on a double circuit dead-end type of towers of suitable design located on the sloping bank of the tail-race.

Alternative Layout III.

A 'low' type of layout with the conventional side-by-side placing of isolators and 'strung' buses. One of the buses of the 132 kV to be in the form of a letter H around the other bus which is straight. The outgoing connections are opposite to the incoming ones as in alternatives I and II. The overhead connections are terminated on a two level gantry at the switchyard in triangular formation thus increasing the phase to phase distance of the conductors.

The alternative layout III is found to be the most suitable amongst the three and has the following advantages:—

(i) The isolators are in a lower level and therefore maintenance and inspection of the same will be easier.

(ii) It is more economical in steel structures.

(iii) The gantry structures for the incoming overhead line connections also serves to take the connections from the circuit breakers to the bus selecting isolators.

(iv) Since this gantry is located in the switchyard itself instead of in the tail race bank as in alternative II, its foundations will not be amenable to erosion from the flowing waters.

SALIENT FEATURES

1. The 132 kV switchyard consists of nine bays of width 32′ 10″ each. The 66 kV switchyard consists of four bays. The dimensions of the switchyard are:— 105m×66m of the 132 kV portion, 66m×37.5m of the 66 kV portion and 131m ×86m overall dimensions.

2. Galvanised steel structures of the lattice type are provided.

3. The maximum tensions of conductors for the incoming line connections, outgoing line connections, the main and jack buses and shielding earth wires have been fixed as below after taking into consideration various factors such as strength of wires and insulators, limitations of tensions at normal temperatures to avoid vibration in wires, electrical clearances, avoidance of unsightly sags at maximum temperatures, and economy of supporting structures:—

(i) Incoming Line connection	7000 lb.
(ii) Outgoing Line connection	3000 lb.
(iii) Main and Jack buses	1500 lb.
(iv) Earth wire:—	
(*a*) For incoming Circuits	3400 lb.
(*b*) For outgoing Circuits	3000 lb.
(*c*) For Switchyard	1500 lb.

By a judicious selection of the above it has been possible to effect considerable economy in design of

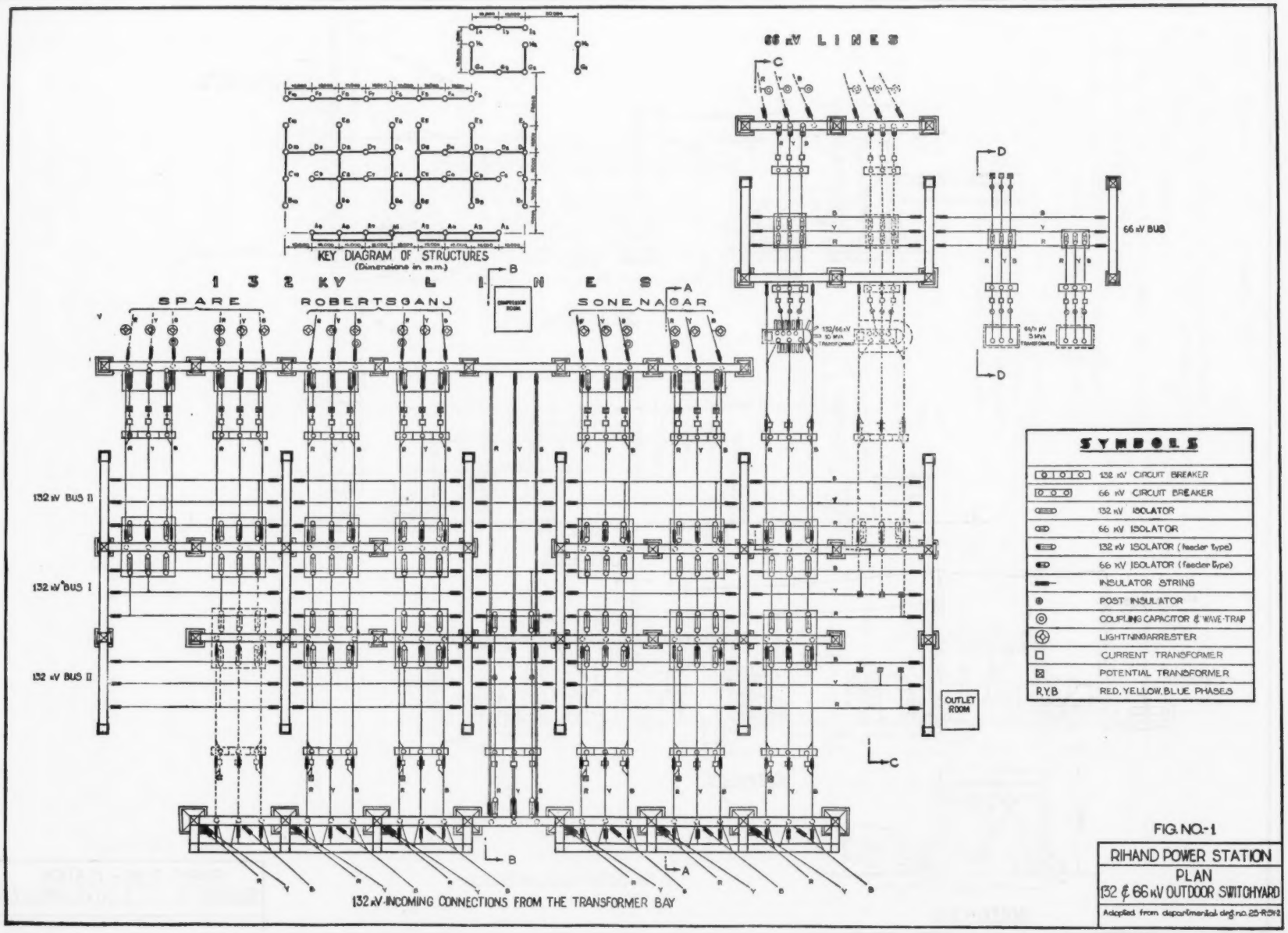


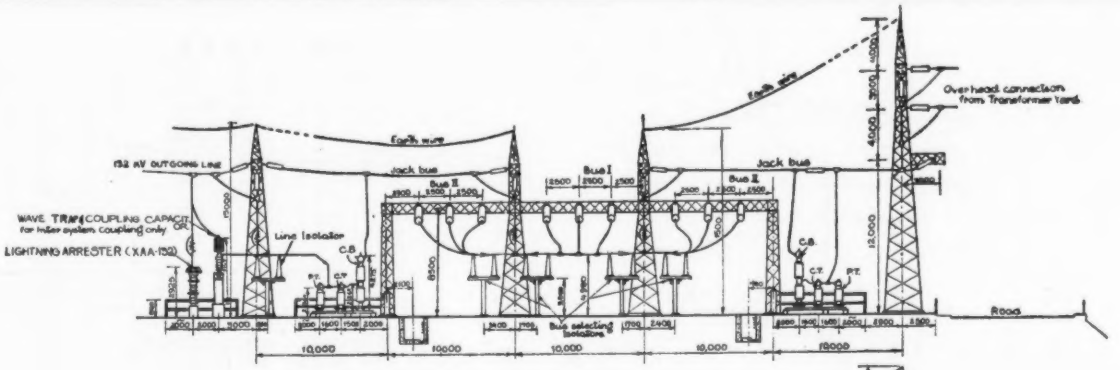
FIG. NO.-1

RIHAND POWER STATION

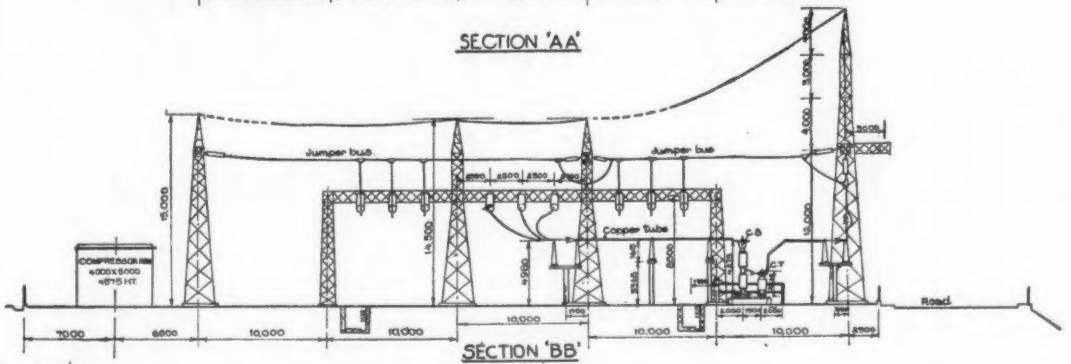
PLAN

132 & 66 kV OUTDOOR SWITCHYARD

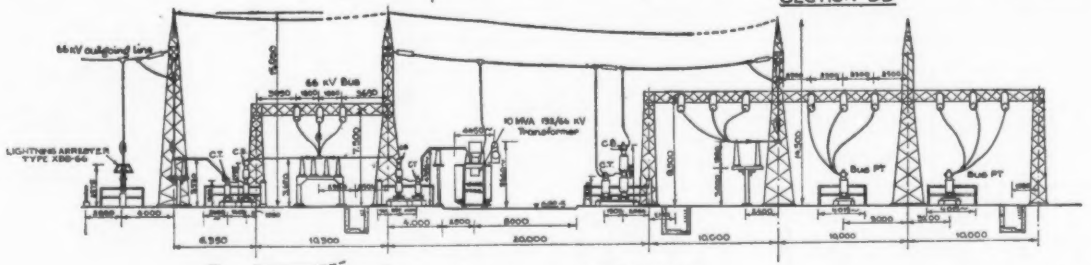
Adopted from departmental drg. no. 25-R-5142



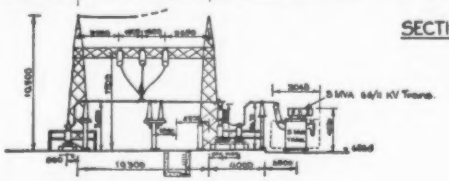
SECTION 'AA'



SECTION 'BB'



SECTION 'CC'



SECTION 'DD'

ALL DIMENSIONS IN MILLIMETERS

FIG. NO.-2.

RIHAND POWER STATION
SECTIONS OF 132 & 66 kV SWITCHYARD

Adopted from departmental drawing 50-R-51

structure. The total weight for incoming and outgoing line gantries, 132 kV and 66 kV bus structures, trestles etc., is 148 tons approximately.

4. For the design of the structures the following criteria has been considered:—

(*a*) Horizontal wind pressures on flat surfaces such as structural members including angle sections —15 lb/sq. ft.

(*b*) For lattice structures, the actual exposed area of one of the latteral face to be increased by 50% and full wind pressure ignoring the shielding effect from other structures in the switchyard.

(*c*) Horizontal wind pressures on cylindrical surfaces such as conductors, ground wires, insulators and such other electrical equipment to be 10 lbs. per sq. ft.

(*d*) Maximum and minimum temperatures of 150°F and 32°F without ice.

(*e*) Factors of safety based on the crippling load of struts and the elastic limit for tension members under the above conditions be not less than 2.5 for supports as well as for their foundations against overturning.

(*f*) The following formulae have been used to determine the unit compressive stresses in lbs/sq. inch on the gross area of section of struts:—

(i) $45000-190\frac{1}{r}$ but not more than 36000 lbs/sq.inch.

for $\frac{1}{r}$ less than 150.

(ii) $36000-130\frac{1}{r}$ for $\frac{1}{r}$ between 150 to 200.

For tension members an ultimate strength (at yield point) of 36000 lb/sq. inch on net area has been taken.

5. Minimum oil circuit breakers of the expansion type, manufactured by CKD, Prague, Czechoslovakia are provided for both 132 kV and 66 kV. The 132 kV breakers are of 3000 MVA rupturing capacity and can be used for single pole or 3 pole reclosures as may be necessary. The operating time of the breakers is 3—5 cycles. The 66 kV breakers are of 1500 MVA rupturing capacity and can be used only for three pole reclosures.

6. The current and potential transformers, insulators, bus bar and other switchgear equipments of the outdoor switchyard are all being supplied by Messrs Skoda and are manufactured in CKD Works, Prague, Czechoslovakia. The current transformers are of the one ampere secondary type and contains two separate cores, one for measurement and the other for protection. Independent P.Ts have been provided for such outgoing 132 kV line and a separate set of V.T. for each of the 132 kV buses.

7. Concrete walk-ways in front of the circuit breakers and other apparatus in the switchyard have been provided for easy access. Cable tunnels and ducts for the control and power cables, compressed air piping have been suitably located inside the switchyard.

CIRCUIT BREAKERS—RATED VOLTAGE, RATED INTERRUPTING CAPACITY AND IMPULSE STRENGTH

While it may be justifiable to adopt a basic impulse level of one step lower for the transformers, reduced insulation has not been applied to the switchgear of Rihand Power Station mainly due to the considerations of Service Voltage. The Service Voltage at Rihand 132 kV bus according to the Net Work Analyser Studies may vary from 132 kV to 145 kV when the Station is supplying the fully loaded Grid, therefore a circuit breaker of a voltage class lower than 132 kV is clearly unsuitable though it may satisfy the interrupting requirements. The impulse withstand value of 630 kV crest has been adopted in respect of the above.

The rated interrupting capacity of 3000 MVA for the 132 kV breakers and 1500 MVA for the 66 kV breakers have been found to be ample from the network studies for the system.

INSULATION OF THE BUSES

The buses and the main taps and connections in the outdoor switchyard are of the 'Strung' type. In choosing the number of discs in the insulator chains the effects of pollution from frequent dust storms occurring in these parts in summer has to be considered.

For a 132 kV system a nine unit string of $10'' \times 5\frac{3}{4}''$ pin and cap discs will meet the impulse requirements, but the creepage distance is only 110″ of which 74″ are protected. To provide more margin for creepage ten units will be suitable for suspension strings giving 122″ total creepage with 82″ protected. For tension strings which will have their axes more or less horizontal an additional two discs would serve the purpose.

For the overhead connections from the transformer bay to the switchyard double strain yokes with strings of 12 units in each are adopted for providing greater mechanical strength.

LIGHTNING ARRESTER RATINGS AND LOCATION

The lightning arrester constitutes the ideal protective device against damages due to lightning strokes. A suitable protection scheme cannot be said to exist unless there is available a reasonable margin between the level of protection provided by the lightning arrester and the basic insulation level of the equipments to be protected. It is also necessary to establish a lower limit of protection which is imposed by

the magnitude of internal voltages which in turn depends upon the system operating conditions, the network itself, system neutral grounding conditions, efficiency of earthing etc. For the solidly earthed neutral system of 132 kV a rated voltage of 120 kV for the lightning arresters would be correct. Also for the protection of power station equipments, 'station type' of arresters having large discharge capacities are desirable from the point of view of offering protection against heavy lightning strokes in the vicinity of the power station.

The lightning arresters selected for Rihand Power Station for the 132 kV and 66 kV are of ASEA make and satisfy the above mentioned requirements. They have the following characteristics.

Serial No.	*Details*	*L.A. for* 132 kV	*L.A. for* 66 kV
1.	Type and make	ASEA	ASEA
2.	Rated voltage (EMS)	120 kV	66 kV
3.	Spark-over voltage at power frequency (Min) RMS	216 kV *	119 kV *
4.	Spark-over voltage at Surge (1/50 kV crest) RMS	250 kV *	145 kV *
5.	Spark-over voltage at steep wave front (Crest)	400 kV *	205 kV *
6.	Discharge voltage at surge 10 kV wave 10/20 (crest)	348 kV *	226 kV (*)
7.	Discharge voltage at surge 5 kV wave 10/20 (crest)	312 kV	202 kV
8.	Maximum discharge capacity Crest	150 kV	100 kV

*$\pm$10%
(*)$\pm$ 5%

Efficient protection of the larger type of substation like the outdoor switchyard at Rihand where a large number of high voltage lines are taken out is possible only when each of the line is provided with a separate set of lightning arrester. These arresters prevent excessive stress from voltage gradient and also secure that when the line breakers or switches are open constituting dead-end reflection points, there will be no flashover across the open contacts or from the line to ground. Separate lightning arresters have been provided for power transformers since they are located away from the switchyard.

The lightning arresters for the 60 MVA transformer at Rihand Power Station are located in a bracket on its body and the lightning arresters for the outdoor switchyard are mounted at the point of entry of the transmission lines into the switchyard.

SHIELDING AGAINST DIRECT STROKES

For proper protection of the outdoor station equipments the lightning arrester protection shall be supplemented by reliable shielding of the station and the transmission lines against direct strokes of lightning. The general type of shielding to be used will of course largely depend upon the general design of the Substation structures. For the Rihand outdoor switchyard, the protection offered by the steel structures has been supplemented by the overhead ground wires wherever essential by connecting the ground wires between top of some structures in zig-zag pattern. Inside and outside shielding angles of 30° and 45° are found to offer adequate protection for all the equipments of the switchyard.

The structures of the switchyard and the equipments of the switchyard shall be properly connected to a low resistance grounding mat provided for the switchyard which in turn will be interconnected with the Power House ground mats one in the upstream side of the dam adjacent to the power house and one in the tail race adjacent to the switchyard.

INSULATION COORDINATION

Insulation coordination is a quantitative co-relation of the insulation of the apparatus and circuits to the overvoltages that will be allowed by the protective methods employed. In the substation protection it essentially involves a comparison of apparatus impulse withstand characteristics with that of the impulse protective level of lightning arresters or supplementary protective methods.

The graph shown in the elsewhere shows the scheme of coordination for the outdoor switchyard at Rihand Power Station.

CLEARANCES

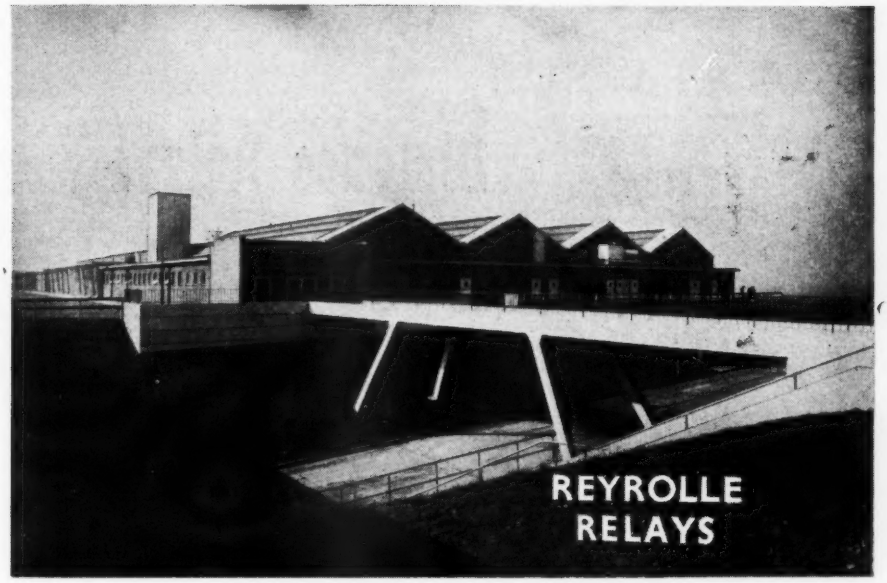
The following minimum clearances between live parts and earthed structural members have been adopted and the design of structural members shall be made permitting these minimum clearances, after allowing for the deflections of the strings etc., if any.

132 kV	..	..	..	54″
66 kV	..	..	..	30″

ERECTION OF THE SWITCHGEAR

The department is preparing all the detailed construction drawings connected with the installation of the outdoor switchyard and it is the endeavour to erect the switchgear equipments departmentally.

I gratefully acknowledge the help of my colleagues in the design office in preparation of this article.



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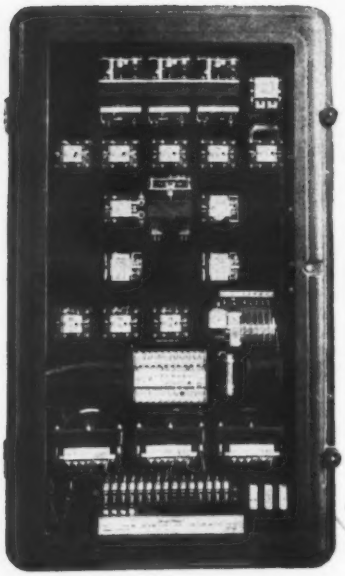

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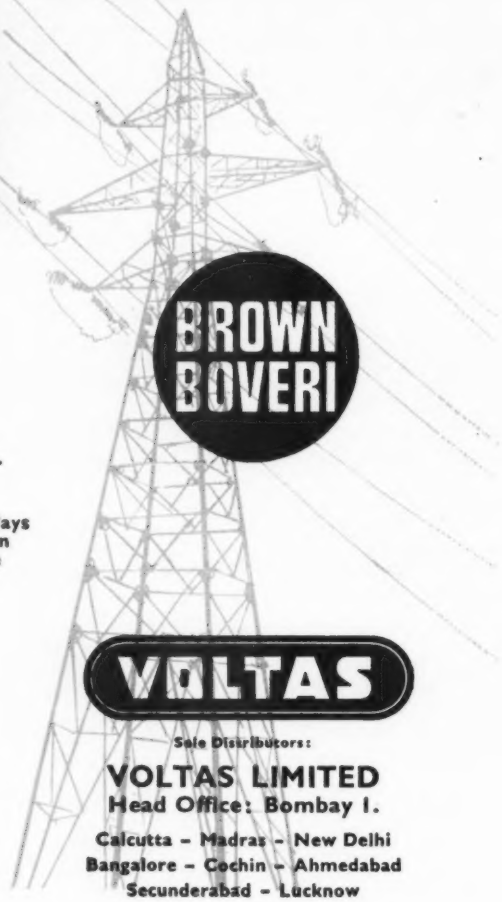

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Power Line Carrier Communication System for Rihand Grid

By H. S. SAWHNEY

WITH the day to day expansion of the High voltage Power Systems, the importance of Power Line Despatching Service for technical control and operational efficiency has become very evident. A reliable telephone system for the verbal exchange of intelligence and a fast operating telemechanism for receiving and recording of important values from different parts of the grid is an essential feature. The P & T lines with their intervening exchanges as a medium of communication and not being entirely at the disposal of the Power Authorities cause considerable dealy. They frequently get out of service in bad weather when their need is the greatest to the Power Authorities. These considerations coupled with the need of high speed simultaneous isolation of feeders at both ends have warranted an independent communication system exclusively for the Rihand Grid stretching over 16 Districts of Uttar Pradesh.

The Power Line carrier communication system is being devised to meet the following requirements:—

1. Telephone communication.
2. Telementering of values from four important Centres namely Pipri, Robertsganj, Mirzapur and Mau to the Centrol office at Varanasi.
3. Teleprotection.

TELEPHONE COMMUNICATION

There are two principal methods of Carrier Communication over Power Lines. These are the wave change-over and the fixed frequency methods. The wave change-over method is characterised by the fact that a group of four or five sections can be covered by only one pair of frequencies while in the fixed frequency method each line section between two stations requires a separate pair of frequencies. The characteristics of these two methods are:—

Serial No.	*Particulars*	METHOD	
		Wave Change-over	*Fixed frequency*
1.	Initial cost	Low.	High.
2.	No. of frequencies required.	Two frequencies for three to four sections	Two frequencies per section.
3.	Use of super-imposed channels.	Not possible	A number of telemetering or other channels could be be super-imposed
4.	No. of stations between which simultaneous conversation could be carried on.	Two.	More than two.
5.	Quality of reception	Comparatively low.	Very High

Keeping in view, efficiency and economy, the importance and requirements of the substations a combined fixed frequency and wave change-over method has been adopted for the Rihand Grid.

The accompanying drawing shows the circuit diagram of Rihand Carrier Communication scheme. The sections marked in red (Pipri-Robertsganj, Robertsganj-Varanasi, Varanasi-Mirzapur, Mirzapur-Sultanpur, Mirzapur-Allahabad and Mirzapur-Pipri) form the fixed frequency districts while the groups Varanasi-Mau-Dhorighat-Ghorakhpur and Sultanpur-Sohwal-Faizabad-Gonda form two wave change-over districts.

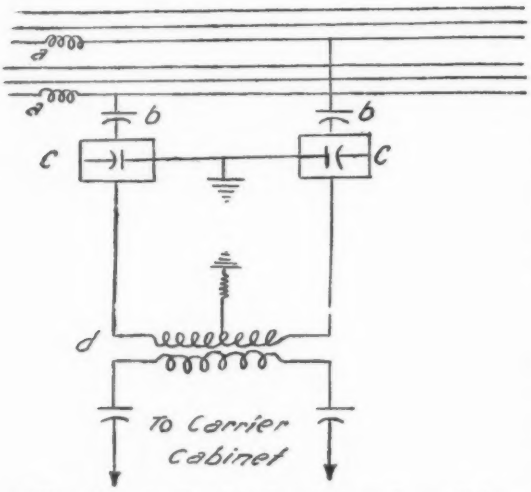
The wave change-over districts have 66, or 11 kV transmission lines except Varanasi-Mau 132 kV. Section. Heavy and important telephone traffic is not expected in these districts. Actual traffic measurements on some Power Line Carrier systems have shown that more than 90% of the telephone calls are conducted between the Power Controller's office and Power Houses or vice-versa whereas less than 10% of the traffic is attributed to calls between the individual substations or Power Houses. Further as explained later, the telemetering and teleprotection devices in these districts do not make use of any super-imposed channels and, therefore, the wave change-over method was judged to be the most suitable.

The fixed frequency districts cover the 132 kV. lines forming the important parts of the grid with heavy and important traffic. In these districts reliability and efficiency has been obtained by taking advantage of the fixed frequency method to provide the frequency shift protection channels to operate in conjunction with carrier telephone cabinets within the telephone band itself.

COUPLING CIRCUIT

Inter-system method of coupling has been employed throughout except at Dhorighat and Faizabad where there is no provision for carrier protection channels and single phase coupling, is equally suitable. Following is the circuit diagram for intersystem coupling.

The Inter-system coupling in the event of a fault in one of the two coupled conductors allows carrier



a—Wave Trap, b—Coupling Capacitor, c—Coarse Voltage Arrestor, d—Line Turning Unit.

current transmission over the non-defective coupled conductor, a resistor having been inserted in the line leading from the centre tap of the line tuning unit to ground, thus making the continuity of communication more reliable. This is highly important when carrier channels are employed for line protection also. A further feature of the inter-system coupling is, that carrier transmission will continue even when all the three conductors of an H. T. circuit are shorted and grounded, a condition commonly met with a repair gang working on the H.T. Line.

CARRIER FREQUENCIES

The Carrier frequencies used lie in the spectrum 150 kc/s to 300 kc/s to avoid interference with the 12 channels P & T open wire carrier telephone systems running in parallel with the H.T. Lines. The P & T carrier systems use the frequency range upto 146 kc/s.

THE EQUIPMENT

The special feature of the coupling equipment is the use of broad band tuned 150-200 or 200-300 kc/s wave traps.

The transmitter and receiver cabinets adopted for Rihand Grid provide full duplex telephone channel transmission employing amplitude modulation on double side band system. Each voice frequency circuit will have a normal band width of 5 kc/s.

The telephone transmission band is from 300 c/s to 2400 c/s. Super-imposed channels have not been provided as the telemetering system is independent and also the protection system works within the telephone band itself and in conjunction with the cabinets for the latter.

The carrier frequency output of the transmitters is 10 watts at 100% modulation.

Four wire interdialing cabinets have been used at Varanasi and Mirzapur to route the calls to the required districts.

Only one channel has been provided on each H.T. line so that one telephone call can be established at a time while in the ring circuit Varanasi-Robertsganj-Pipri-Mirzapur it is possible to hold two conversations simultaneously from the same station, for instance from Varanasi to Pipri (Via Robertsganj) and simultaneously from Varanasi to Mirzapur. A third simultaneous conversation is also possible from Varanasi to Mau, Dhorighat or Gorakhpur.

A priority facility is incorporated in all the telephone sets at the sub-stations, Power Houses and at other important subscribers. By operating the priority key one could enter the conversation already going on. Ofcourse the persons who are in conversation will get an indication that somebody is trying to override.

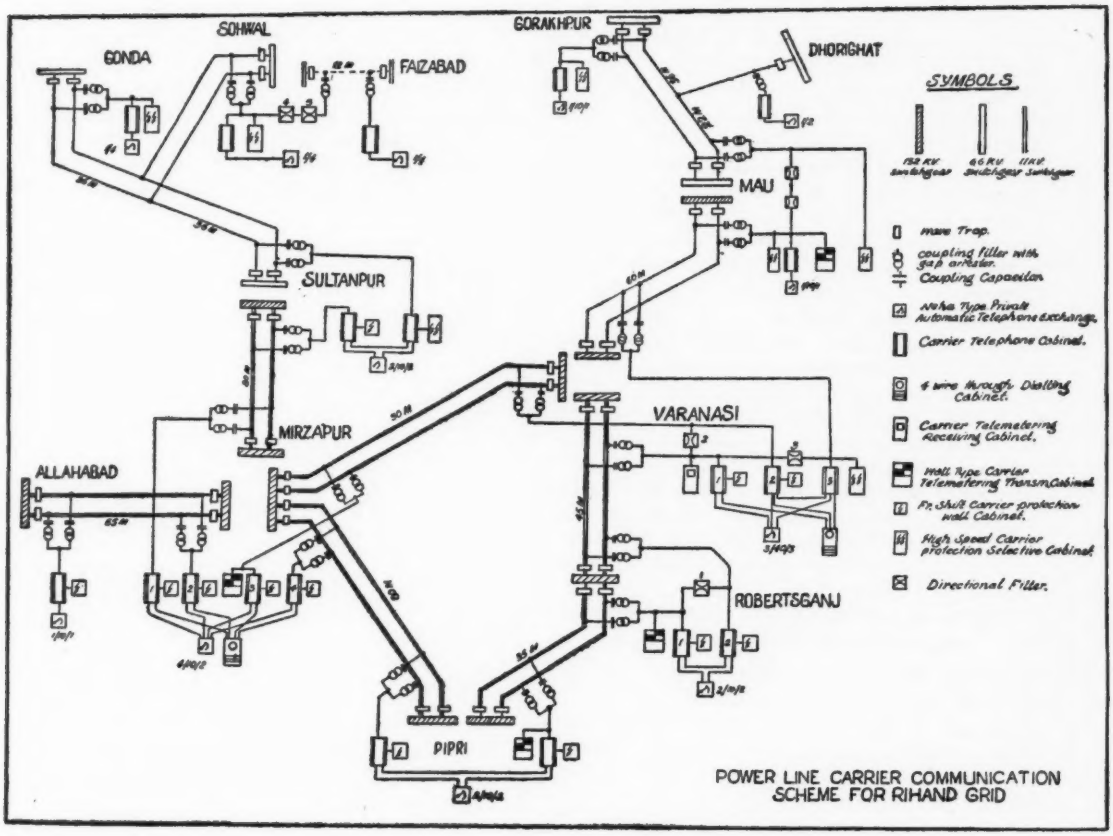
A conference facility has been provided at the Varanasi Control office. Three conference call numbers will be set up for the following conference circuits.

(i) Control office at Varanasi-Robertsganj-Pipri.
(ii) Control office at Varanasi-Mau-Dhorighat-Gorakhpur.
(iii) Control office at Varanasi-Mirzapur-Sultanpur-Sohwal-Gonda.

Any combination of conference calls is possible from a special board arranged in the control office at Varanasi such as over three circuits simultaneously or any two of them. In addition the officer-in-charge of Varanasi Sub-station can also be included in any conference.

WORKING OF TELEPHONE SYSTEM

The dial number scheme has been so devised that any station may be reached by any other station under the same call number. It does not make any difference whether the desired party is connected to the same district as the calling station or whether the call has to be relayed through a number of other telephone districts. Suppose a subscriber A at Robertsganj calls Subscriber B at Mirzapur (through Varanasi). If the line Robertsganj-Varanasi is busy A will hear the busy tone (800 c/s tone sent at a high interruption rate) requesting the calling party to replace the hand set. In case the line Robertsganj-Varanasi is free A will hear a dial tone (the so called proceed to dial signal). This means that the cabinet No. 1 (Varanasi) is ready to receive the dial number of the desired station. On dialling the first digit of the required number A goes over from cabinet no. 1 (Varanasi) to the through dialling cabinet Varanasi and finally to cabinet no. 3 (Mirzapur). If section Varanasi-Mirzapur is free, on further dialling he gets connected to B. If the section Varanasi-Mirzapur is busy the cabinet no. 1 Varanasi will return the transit busy signal. This tone is intended to summon the calling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POWER LINE CARRIER COMMUNICATION
SCHEME FOR RIHAND GRID

party to replace the hand set or to operate the priority key. If the calling party fails to replace the hand set or to depress the priority key, forced release will take place after about 30 seconds and he will receive the normal busy signal. The line is now free for other sections *i.e.*, a call incoming at this moment will be signalised immediately after replacing the handset.

TELEMETERING

The telephone equipment alone is not capable of meeting all the demands of the load despatcher like immediate reports from the important points of the grid. This necessitates the use of telemetering system to give the load despatcher a constant picture of the state of the system at any moment.

An 18 channel multiband telemetering equipment working in separate frequency band of 2.5 kc/s width has been adopted for Rihand Grid. The telemetering system is based on the pulse rate system. The following values will be telemetered at Varanasi-Control office from different stations of the grid.

Name of Power House and sub-station.	Active Power	Re-active Power	Voltage	Reservoir level	Total number
1. Pipri Power House					
(*a*) Generated ..	1	1	—	—	—
(*b*) Reservoir level ..	—	—	—	1	3
2. Robertsganj Sub-station.					
(*a*) Received ..	1	—	—	—	—
(*b*) Sent out ..	1	—	—	—	2
3. Mirzapur Sub-station.					
(*a*) Received ..	1	—	—	—	—
(*b*) Sent to Sultanpur ..	1	—	—	—	—
(*c*) Sent to Allahabad Sub-station ..	1	—	—	—	3
4. Mau.					
(*a*) Received ..	1	—	—	—	—
(*b*) Sent out ..	1	—	1	—	3

This indicates that a total of 11 values in all shall be telemetered at Varanasi. At present the telemetering cabinets at Pipri, Robertsganj, Mirzapur and Mau are equipped with three, two, three and three transmitting channels respectively according to immediate requirement. The remaining channels upto a maximum of 18 can be added easily without incurring much expenditure. The telemetering receiving cabinet at Varanasi control office is equipped with 18 receiving channels.

The telemetering signals from Pipri and Robertsganj find an independent path to Varanasi telemetering receiving cabinet through the directional filter no. 1 located at Robertsganj. Similarly the telemetering HF bands from Mau and Mirzapur find their paths to telemetering receiving cabinets at Varanasi through the directional filters no. 3 and 2 respectively located at Varanasi. In view of the frequency spectrum being overcrowded the telemetering equipment makes use of single side band system which requires only 2.5 kc/s band for each direction.

TELEPROTECTION

Teleprotection system provides tripping signals for breakers at both ends of line for simultaneous ope-operation.

The Rihand Teleprotection System can be divided into three parts as described below:—

(*a*) Three leg carrier protection on Sultanpur-Sohwal-Gonda Line sections.

No circuit breakers have been provided on the 66 kV side of 132/66 kV. transformers at Sultanpur and 66/33 kV. transformers at Sohwal and Gonda. This necessitates a special protection system for effectively protecting the transformers. High speed carrier relaying equipment has been used for this purpose with two transmission channels (double circuit line) for protecting two H.T. feeders or transformers and four receiving channels (each protection cabinet at these stations has to look to the remaining two stations) in two separate H.F. bands for tripping the circuit breakers on the 33 kV. side of the H.T. transformers at Sohwal and Gonda and on the 132 kV. side at Sultanpur sub-station respectively. Special alarm circuit and facilities for easy checking and recommissioning of the carrier circuits have been provided. The speed of operation of protection equipment being 9 to 10 milli seconds ($\frac{1}{2}$ cycle). Because of the high speed of operation of the protection equipment, the simultaneous clearing of faults at both ends can be achieved in the real sense so important for transformer protection.

(*b*) High speed Carrier Protection Equipment for the Line Section Varanasi-Mau-Dhorighat.

The same type of High Speed carrier protection equipment with special alarm circuits has been included for the line sections Varanasi-Mau and Mau-Gorakhpur. The carrier cabinets are, however, equipped for 2 transmission channels and 2 receiving channels only in the H.F. band of 2.5 kc/s band width.

(*c*) For the remaining H.T. Line sections there is adequate provision of circuit breakers etc. and therefore conventional type of carrier protection channels working in conjunction with carrier telephone cabinets within the telephone band itself have been included. These cabinets have two transmitting and two receiving channels each.

The same cabinets are used for fixed frequency or wave change over operation with minor alterations.

In the wave-changeover districts Sultanpur-Sohwal-Faizabad-Gonda the Faizabad substation, an offshoot from Sohwal has been included by the use of

(Continued on page 77)

Scheme of Protection at Rihand Power Station

By MATA PRASAD

THE single line diagram in fig. 1 shows the switching layout of generators, transformers, 132 kV & 66 kV busbars and outgoing feeders at Rihand Power Station. The 50 MW generator is connected to a 60 MVA three phase transformer in conventional unit layout having a solid tee off for a 500 kVA unit transformer for supplying power to its own auxiliaries.

An alternative arrangement for supply to station auxiliaries is provided from the 11 kV side of the 3 MVA transformers through 500 kVA 11/0.4 kV Station transformers.

A combination of differential, impedance, over-current and restricted earth fault protection has been provided for the various equipments as below:

Section (i) 50 MW Generator 60 MVA Transformer (including 500 kVA unit transformer).
Section (ii) 132 kV Busbar.
Section (iii) 10 MVA, 3 MVA and 500 kVA Station Transformers.
Section (iv) 132 kV feeders.

Only salient features of the protection schemes and relays under each section are described below.

NEUTRAL GROUNDING AND EARTH FAULT PROTECTION

The neutral on the 132 kV side of the 60 MVA step up transformer is solidly earthed and the neutral of the generator is earthed through a single phase 30 kVA 6350/110 V distribution transformer. Besides limiting the fault current in the generator transformer section this voltage transformer affords means for earth fault protection for generator and the primary sides of the 60 MVA transformer and 500 kVA unit transformer. The neutral of generator could be left isolated as the 60 MVA 11/132 kV Delta/Star transformer (42 Ydll) has its neutral of the 132 kV winding earthed solidly but due to chances of damage and also mal-operation of earth fault relays from the surges penetrating into generator section through the inter-winding capacitance between primary and secondary windings of the transformer it is considered advisable to stabilize the neutral of the generator by earthing and provide as well surge diverters in conjunction with capacitors at generator terminals.

The secondary of the 30 kVA voltage transformer is shunted by a voltage relay and a loading resistor. (The ohmic value of the resistor has been so determined that the voltage relay does not operate incorrectly due to high voltage, that may appear across generator neutral and ground as a result of ferro-resonance). The purpose of this resistor is two fold; firstly it damps out oscillations set up by ferro-resonance and secondly it allows to circulate 5A to 10A in the generator under earth fault condition.

It may be noted that when generator neutral is earthed through a voltage transformer single phase to ground fault does not produce fault current sufficient for correct operation of differential relay. Therefore a sensitive and stable earth fault protection relay has to be provided.

The voltage relay which is connected across secondary terminals of voltage transformer is an induction pattern inverse definite minimum time (IDMT) relay provided with frequency compensation. This relay provides earth fault protection to generator and primary windings of 60 MVA transformer and 500 kVA unit transformer.

SECTION I: 50 MW GENERATOR—60 MVA TRANSFORMER UNIT

Internal Phase to Phase & Three Phase Faults

The protection schematic for generator transformer is shown in fig. 2. It is a differential protection separate for generator and transformer commonly known as longitudinal differential protection. The zones of the two differential protection overlap each other. Because of inherent inaccuracies of current transformers under high current incidental to a severe fault the differential relays will be biased 5% in case of generator differential and 10% in case of transformer differential. Higher bias in case of transformer differential protection is needed due to the ratio error introduced by the taps on the transformers.

The 500 kVA unit transformer being solidly connected to the generator lies in the zone of generator protection. The provision of current transformers on the primary side of the unit transformer and their connection in the differential circuit of the generator relays may be noted. The zone of protection of generator differential relays will only be extended upto the position of the above current transformers. Faults beyond these current transformers in the primary or secondary winding of unit transformer will be taken care of by over-current and restricted

earth fault protection provided on the transformer.

The differential relays are instantaneous in operation as double or three phase faults are very dangerous from the point of view of damage to machine.

The percentage differential relays are of M/s. English Electric Company induction pattern types DDG & DDGT for generator and transformer differential protection respectively. The pick-up setting range is 5 to 10% for type DDG relay and 10 to 20% for type DDGT relay.

The characteristic of the differential relays are such that under through fault condition the relays are stable.

Transverse Differential Protection for Generators

The longitudinal differential protection cannot detect fault between turns and accordingly for inter-turn protection a separate transverse differential protection on each machine is provided. The stator winding is a four circuit winding per phase but only two terminals per phase are brought out. On each terminal current transformer of ratio 1500/5 is provided and are cross connected. One end of the current transformer is formed into a star point and the other is connected to a definite minimum time, attracted armature type over-current relay of English Electric type CAG 34.

External Faults

For back up against faults outside the zone of generator protection a time delayed over-current relay with IDMTL characteristic type CDG is provided. This relay will be suitably time graded with respect to other over-current or impedance relays provided on the grid feeders. A stand-by earth fault relay is also provided in 132 kV neutral of the 60 MVA transformer for faults on the lines and busbars.

Unbalanced Loading

The damper windings of Rihand Generators are so dimensioned that they can sustain, without undue temperature rise, a negative sequence current of 1450 amps. approximately. Negative sequence current protection for unbalanced loading is not needed because the feeder relays catering for external two phase faults are high speed impedance relays as mentioned later.

Inter-turn Faults on 60 MVA Transformer

When a surge breaks down the inter-turn insulation of a transformer the local short circuit may not be detected by transformer differential protection. The fault current in the short circuit loop is quite high and this at the incipient stage produces sufficient gas due to arcing and local heating in oil, to be detected by bucholz relay. Double float type bucholz relay is therefore being provided on each 60 MVA transformer. One float is for alarm and the other for tripping.

Rotor Earth Fault

Because neither pole of the generator field is normally earthed in service, an earth fault at one point is not dangerous. If a second earth fault develops at some other point in the field it forms a short circuited loop and the field breaker as well as main 132 kV circuit breaker have to be tripped immediately. To avoid such eventuality detection of earth fault in rotor winding is obviously important. The earth fault relay may only give alarm to enable the operator to transfer load to other machines and isolate the faulty machine.

Rotor earth fault relay is a sensitive voltage relay connected to the mid point of a centre tapped resistor across the rotor winding. To carry out a periodic test (which is necessary because a fault in the centre of winding will go undetected) a test push button switch is included which offsets the relay from the mid point of potentiometer and during a periodic check up any such fault could be detected.

Unit Transformer Faults

From fig. 2 it is clear that earth fault in the primary of unit transformer will be detected by generator earth fault relay. The phase to phase fault on the transformer primary winding will be taken care of by overcurrent relay provided for unit transformer overload protection as this does not come under generator differential protection zone.

For earth and phase faults in L.V side of unit transformer restricted earthfault and overcurrent relays are provided as shown in figure 2. Thus full protection for phase and earth fault in unit transformer has been provided. It may be noticed from figure 2 that L.V. circuit breaker is interlocked with H.V. circuit breaker such that when H.V. circuit breaker is tripped it also simultaneously opens the L.V. circuit breaker. This prevents feeding to internal fault from other generators through the L.V. circuit.

Figure 4 is a block diagram showing protection shutdown scheme. The drawing is self explanatory.

SECTION II: 132 kV BUSBAR

Now-a-days practically all the high voltage busbars at the generating stations are being provided with some form of busbar protection. The busbar faults are mainly due to:—

(i) Flashover of insulation and breakage of insulator disc. This may be due to inherent weakness of material or heavy contamination of atmosphere.

(ii) Flashover of insulation due to lightning surges.

(iii) The other cause may be due to human errors in operation and maintenance.

As said earlier at Rihand Power Station the double busbar layout has been adopted. The two busbars are coupled by a bus coupler. The busbar protection

at this sub-station consists of impedance relays working in conjunction with time relay in all the incoming circuits and an overcurrent relay on the bus coupler.

The impedance relays and time limit relays are of Messrs. Brown Boveri types ZA3 and MLT respectively. The impedance relay has got two coils, current coil and voltage coil. The current coil receives its supply from 300/1 protective C.T. in the incoming circuit and voltage coil gets its supply from busbar P.Ts through bus selecting switches. The characteristic curve of this plain impedance relay when plotted on R-X plane is a circle whose centre coincides with the origin, the seat of relay. Thus the relay is non-directional and will see on both sides. These busbars are also under second zone setting of distance relays at Robertsganj and Sonenagar ends. The second zone clearance time is 0.4–0.5 sec.

The distance setting of the impedance relay is 60% of first zone setting of Pipri Robertsganj Section. This comes out to 17 miles which means that for a fault within first 17 miles from Pipri both the feeder protection as well as busbar protection relays will see the fault (assuming one machine on bars). The feeder protection will trip instantaneously but busbar protection will not. The reason being that the busbar protection relays do not trip directly. They close their contacts for fault within their reach and energise their time limit relays (one with each impedance relay) which close their contacts to trip the incoming breakers. If for a fault on line within the reach of bus protection relays, the outgoing feeder breaker does not get trip impulse or fails to trip due to any reason whatsoever, the bus protection relays after a time delay of 0.5 sec. trip on the incoming circuits. At the same time if there be any infeed from the distant substations such as Robertsganj or Sonenagar the breaker at those sub-stations will trip in second zone time.

As the bus protection relay is non directional it protects some percentage of star winding of 60 MVA transformer. Here again discrimination is obtained by time grading. All the transformer faults are cleared practically instantaneously whereas the bus protection relays trip after 0.5 sec. time delay.

In the discussion uptil now only one machine was considered on bars. The actual reach of these relays beyond the busbars will be considerably less than the normal setting if there are more than one machine on bars. For example, each relay will be set for 17 miles of feeder length. When five machines are supplying the load equally and a fault occurs on the line say at the fifth mile on Robertsganj Feeder the relays will not be seeing the fault because the reach of the relay has been reduced to approximately 1/5 that is 3.4 miles only. This is an advantage in limiting the reach of such relays to a very short distance beyond the busbars.

This type of bus protection relay is very popular and has the following advantages:

1. Cheap, does not require any extra C.T. It may be noted that this protection was 20 times cheaper than differential protection including special C.T's.
2. Operation reliable: Circuit very simple.
3. Operation time is only 0.5 sec. which though high for stability limit is short enough to limit damage to the outdoor sub-station.
4. Proper discrimination under heavy through fault current is maintained.

Over-Current Protection of Bus Coupler

An overcurrent relay (definite minimum time, the normal setting is 0.3 sec.) for protection of bus coupler has been included in the bus protection scheme. When both the buses are coupled through the bus coupler, and a bus fault occurs the faulty bus is disconnected from the healthy one in 0.3 sec. The breakers connected to faulty bus is tripped after 0.5 sec. and the supply maintained through the healthy bus.

SECTION III: 10 MVA, 3MVA AND 500 kVA STATION TRANSFORMERS

Figure 3 shows the protection schematic for these transformers. As usual, the protective relays provided for 10 MVA transformer are percentage differential, buchholz and over current (on the primary side of the transformer). The temperature alarms etc. have also been provided.

Differential Protection of 10 MVA Transformer

As the transformer is of vector group 11 Yyo the current transformer on both sides have to be connected in delta. A percentage differential relay of type DDT of M/S English Electric having three different bias slopes 20% 30% and 40% and setting adjustable between 40 and 120% ha_s been provided.

The relay is of induction type having two electromagnets and a common disc. The operating electromagnet produces a torque in the contact closing direction and the restraining magnet in the contact opening direction. The restraint coil is energised by the through current which circulates between the two C.T's and the operating coil is energized by the difference between two C.T. currents. In this way the current required to operate the relay increases as magnitude of through fault current increases, giving the relay a high stability on heavy through faults.

The bias slope of 20, 30 and 40% are the minimum obtained with minimum pick-up setting of 40%. The percentage slope for any bias tap increases with increase of pick up setting. For example a relay set on the 20% bias tap will have an actual bias of 35 percent with a pick up setting of 100%.

The range of time setting is sufficient to prevent inadequate operation due to inrush of magnetising current.

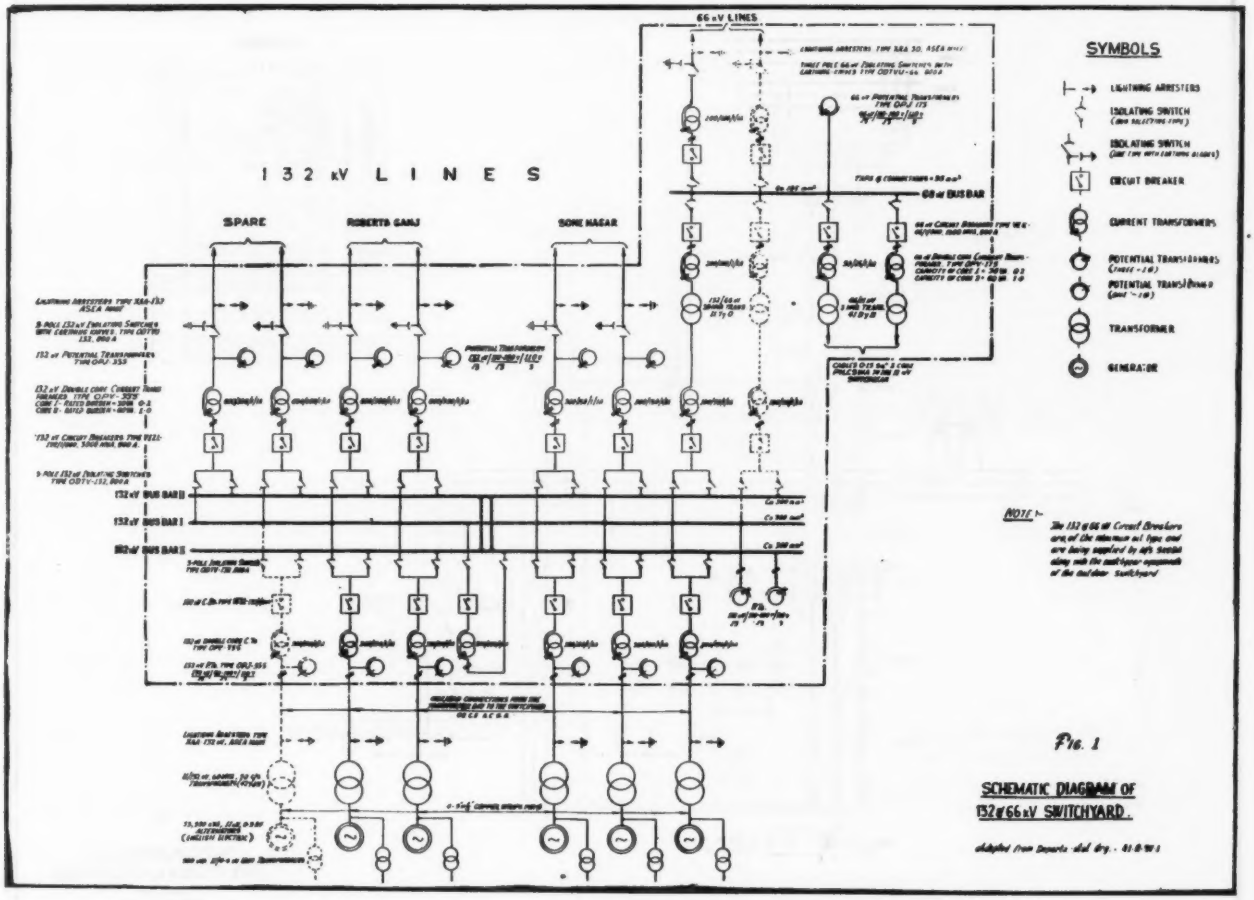


Fig. 1

SCHEMATIC DIAGRAM OF 132 & 66 kV SWITCHYARD.

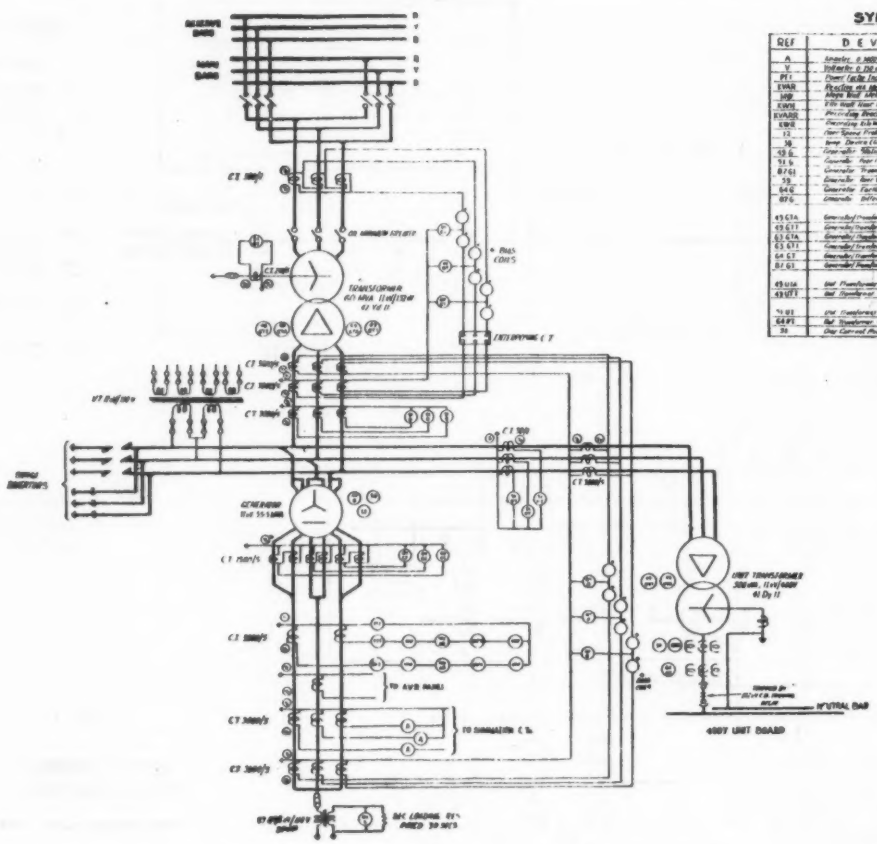


Fig 2

PROTECTION SCHEMATIX FOR
50 MW GENERATOR @ 60 MVA TRANSFORMER

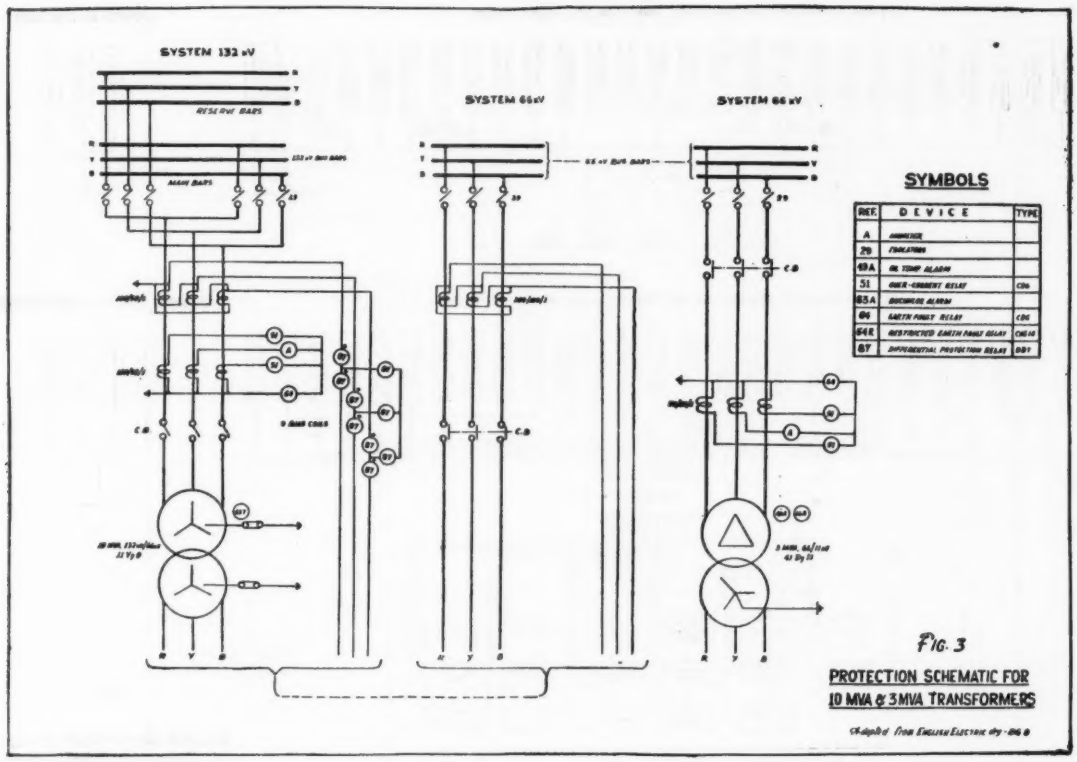
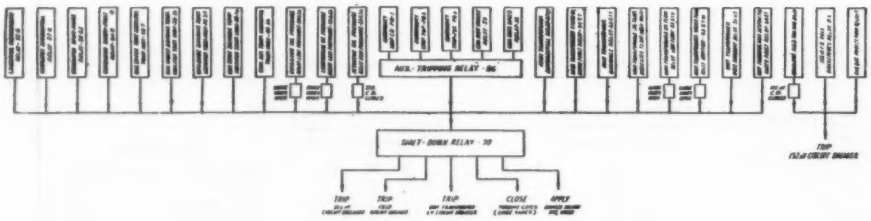


FIG. 3

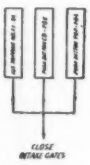
PROTECTION SCHEMATIC FOR
10 MVA & 3 MVA TRANSFORMERS

Adapted from English Electric dwg - 846 B

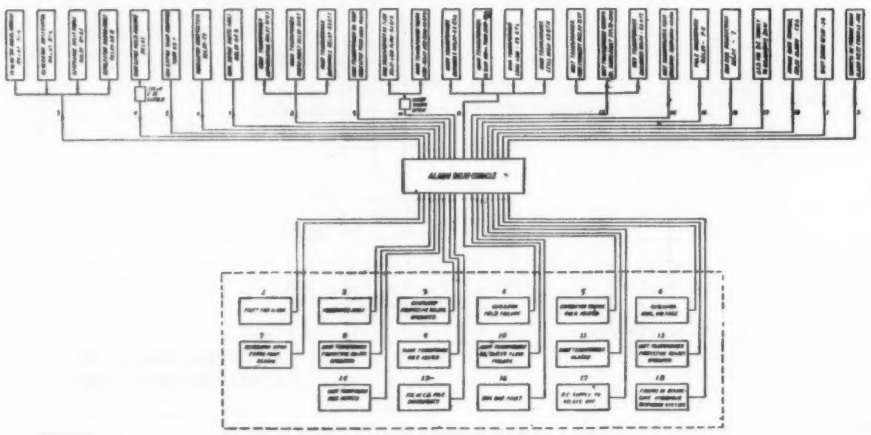
55·5 MVA GENERATOR-TRANSFORMER-UNIT PROTECTION SHUT-DOWN SCHEME



OPERATION OF INTAKE GATES



55·5 MVA GENERATOR-TRANSFORMER-UNIT TROUBLE ALARMS & FACIA INDICATIONS



STC FACIA

55·5 MVA GENERATOR FIRE PROTECTION SCHEME

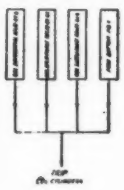
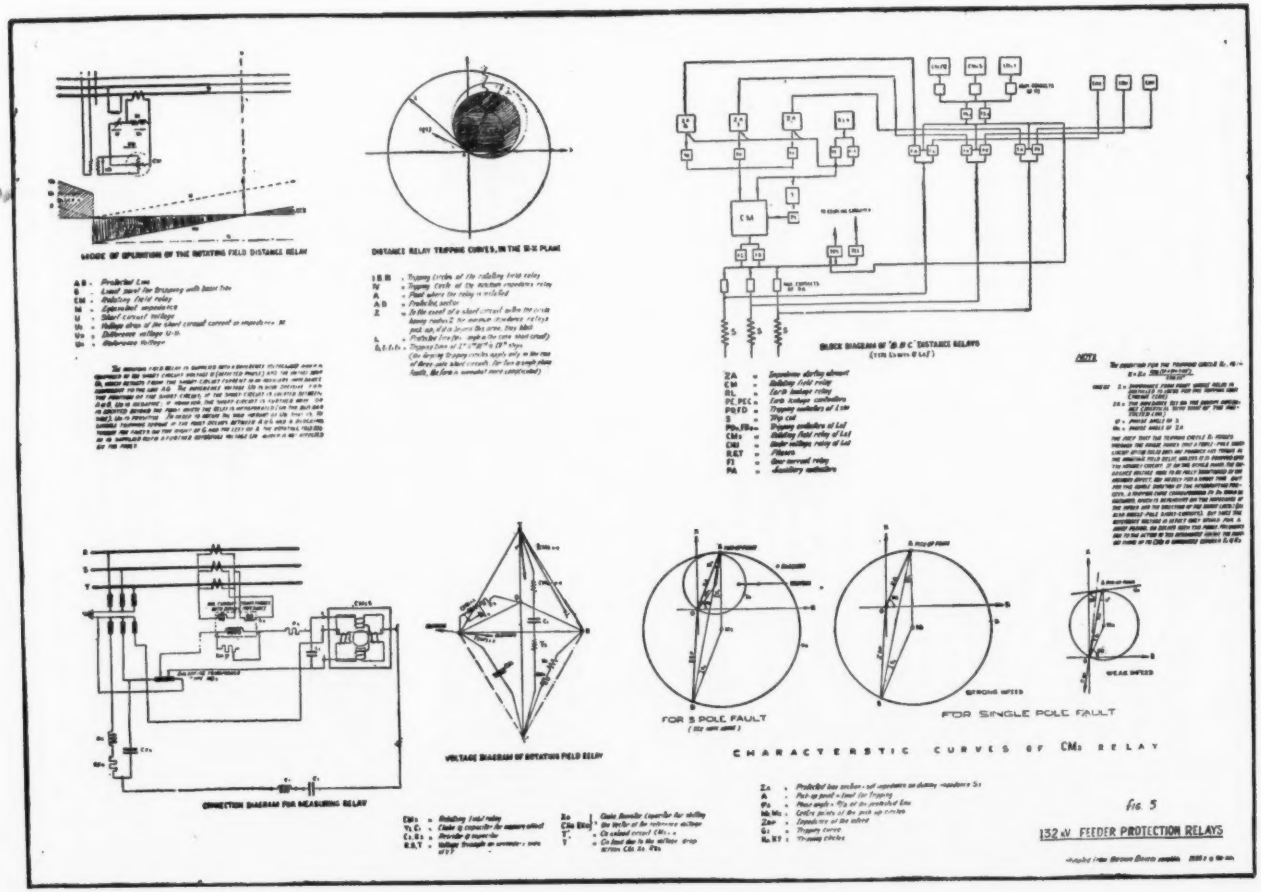


Fig. 4

GENERATOR TRANSFORMER UNIT PROTECTION SHUT-DOWN & ALARM INDICATION SCHEME.

Adapted from Superseded Drawing - 72-Q-LD-1


MODE OF OPERATION OF THE ROTATING FIELD DISTANCE RELAY
DISTANCE RELAY TRIPPING CURVES, IN THE R-X PLANE
BLOCK DIAGRAM OF 'B.B.C' DISTANCE RELAYS
(FOR 3 UNITS OF LAT)
CONNECTION DIAGRAM FOR MEASURING RELAY
VOLTAGE DIAGRAM OF ROTATING FIELD RELAY
FOR 3 POLE FAULT
FOR SINGLE POLE FAULT
CHARACTERSTIC CURVES OF CM3 RELAY
FIG. 5
132 kV FEEDER PROTECTION RELAYS

Over-Current Back up

A combined overcurrent current and earth fault (I.D.M.T.L.) relay of type CDG of Messrs. English Electric Company on the primary side of transformer provides the back up protection.

Protection of 3 MVA Transformer

The differential protection has not been provided on this transformer. The 66 kV side (delta connected) is protected by means of combined phase and earth fault relay of type CDG of Messrs. English Electric. Co. The 11 kV side is being protected by restricted earth fault protection. The 11 kV cable connecting the transformer and 11 kV switchgear comes under R.E.F. protection zone.

Oil temperature and buchholz alarms have also been provided.

Protection of 500 kVA Station Transformers

These transformers are protected by over current relay on 11 kV side and restricted earth fault relay on L.V. side.

SECTION IV: 132 kV FEEDERS

On all the feeders of the Rihand Grid high speed impedance relays with carrier have been provided. A network analyser study was carried out for the Rihand system to determine the optimum time of operation of breaker from stability point of view. It was found that to maintain stability fault has to be cleared within 7.5 cycles from both ends of the line (The operating time of the 132 kV breaker is 3-5 cycles therefore the relay operating time is 2.5 cycles at the most). That meant that carrier accelerated tripping for end zone faults was necessary. From Rihand Power Station busbars, at present, one D.C. line is taken to Moghal-Sarai via Robertsganj and one D.C. line to Sonenagar for railway electrification. The Sonenagar feeder is 100 miles in length and the Robertsganj feeder only 35 miles. The feeder protective relays for all the feeders of Rihand Grid are being supplied by Messrs. Brown Boveri Ltd., Switzerland.

The protective gear comprises of one cycle single step distance type L6f plus a three cycle four steps distance relay, type L3wys or L3wyas depending upon whether the line length is less than 60 miles or more than sixty miles.

These relays (type L3) are conventional step distance relays having a starting relay and a measuring relay. In the relay type L3wys there are three starting relays one for each phase and one measuring relay which caters for all the three phases and all the four steps for all types of faults.

One cycle relay has one measuring relay for each phase permanently connecting to C.T's & P.T's and therefore does not need any starting relay. Under voltage relay type CHUc are provided, instead, for phase selection. This one cycle relay is meant to clear only single phase to ground and three phase faults within the first zone, whilst zone one of three cycle relay acts as back-up. For phase to phase faults, three cycle relay is available.

A brief description of the relays under the scheme is given below.

RELAY TYPE L3wyS

Minimum Impedance Starting Relays

There are three impedance relays, identical in characteristic, one for each phase. The impedance relay is non-directional and is connected for detecting the fault. The difference between L.3wyas relay and L.3wys relay lies in the starting relay only. In the former the starting relay is slightly offsetted towards the direction of distance measurement to reduce the area under the characteristic curve on R-X plane for line length more than 60 miles, while in latter it is plain impedance.

As the measuring relay is normally disconnected the function of the starting relay is under inception of fault, to connect proper current and voltage to the measuring relay. The reach of the relay is governed by following points:—

1. Starting relay of adjacent zones must fall back when the fault is cleared.

2. With double circuits line when one circuit is out the other carries a higher operating current. This must not disturb the reach of the relay.

3. In case of single phase to ground fault on one phase of a circuit (double circuit line assumed) sufficient current may be induced in the healthy circuits due to electromagnetic coupling. The mutual coupling factor for a D.C. line may be as high as 0.6. It is required that relays of healthy phases must not be working.

4. The reach should cover at least two sections, the sections being protected and the section adjacent to it, plus 15% to account for error in measurement. This is important because the starting relay provides back-up in fourth step.

Measuring Relay

In relay L3wys there is only one relay catering for all types of faults, for all the three phases and for the first three steps. The measuring relay is named as rotating field relay and is based on MHO principle. The characteristic of the relay on R-X plane is a circle passing through origin. Thus distance measurement and directional differentiation are done by a single relay. The connection of voltage and current circuits are based on standard practice that is for faults other than single phase to ground faults, the line voltage and difference of current called delta current are supplied to voltage and current coils. In case of single phase to ground fault the voltage coil receives

phase to neutral voltage and current coil gets phase current plus a fraction of totalised current. With this arrangement for all types of faults, the rotating field relay measures positive sequence impedance upto the point of fault.

As the same relay caters for all the three phases and steps, the switching of proper measurands to coils of measuring relay is done by auxiliary contactors. The phase selection is done by the starting relays. It may be noted that the measurement of distance is not affected by arc resistance because the reference voltage which feeds the measuring relay is taken from the healthy phases and is chosen very nearly in phase coincidence with estimated direction of voltage drop across the resistance of a possible arc. The working principle is explained in fig. 5 which shows also the tripping curves of the L3wys and L6f relays.

Reclosing

This relay L3wys (and also L6f) in conjunction with a reclosing relay type OZ110 supplied by Messrs. Skoda, the suppliers of Switchgear is capable for single or three pole reclosing. A reclosing selector switch having four positions is provided in the distance relay. This permits the following service conditions:—

Position 1.

Single and three pole reclosing that is single phase tripping for single phase to ground faults and three phase tripping in case of double or three phase short circuits.

Position 2.

Three phase reclosing.

Position 3.

Single phase reclosing that is single phase tripping in case of single phase to ground faults and three phase final tripping in case of 2 or 3 phase short circuits.

Position 4.

Operation without reclosing.

The 132 kV circuit breakers of this Power Station as well as the grid sub-stations are of single pole type and are suitable for single or three pole reclosing.

One Cycle Distance Relay Type L6f

As said earlier the tripping time of L3wys relays is three cycles (under certain conditions) and this time when added to breaker operating time of 5 cycles gives a total clearing time of 8 cycles whereas for stability purpose the maximum fault clearing time is 7.5 cycles. This necessitates the provision of a relay having a still shorter operating time. To meet this requirement, a single step one cycle distance relay has been provided.

This relay is for single phase to ground and three phase faults. Therefore the voltage coil is supplied with phase to neutral voltage and the current coil with phase current plus a fraction of totalized current.

Phase to phase faults within 50% may also be detected but owing to the connections employed for the measuring elements the distance measurement is not always accurate.

Working Principle

As previously stated the relay L6f has three measuring relays one for each phase. The design and mode of operation of the measuring relay of L6f is basically the same as that of measuring relay of L3wys as explained in fig. 5. The distance measurement and directional differentiation is done according to same principle.

The main difference between the measuring relay type CM3 of L.6f and measuring relay type CM of L3wys is that where latter is not always connected to the instrument transformers except at time of fault by the action of starting relays and its contactors, the former, relay type CM3 is always connected to instrument transformer to have very short operating times. The second difference is in the polarizing voltage circuit and provision of memory circuit. The latter is not provided in the L3wys relays.

It is needless to say that every distance relay working on MHO principle has got three input quantities viz., operating current, restraint voltage and polarising or reference voltage. The polarizing voltage can be taken from the same phase as that of restraint voltage or from the healthy phases. In the relays L6f & L3w polarizing voltage has been obtained from healthy phases. The main advantage is that for nearby interphase faults the polarizing voltage does not vanish otherwise memory circuit to keep the voltage to required level has to be added. The main objection to memory circuit is that it forms a sort of oscillatory circuit and due to variation in supply frequency there results a phase shift between polarizing and faulty phase voltage, causing thereby error in measurement of distance.

The disadvantages of taking reference voltage from healthy phases are:—

(i) For three phase terminal fault the reference voltage vanishes completely.

(ii) Under many conditions of plant it is difficult to draw the characteristic curve on R-X plane under single or double pole faults.

In this relay CM3 of L6f following modifications to remove these disadvantages have been made:—

(i) To cater for 3 phase faults memory circuit has been added.

(ii) A reference voltage composed of two linked voltages which produce a resultant voltage corresponding to the positive sequence component of the phase voltage is taken.

(Continued on page 102)

Rihand Power Station Layout

By ATMA RAM GUPTA

RIHAND Project is primarily for power generation. The live storage capacity of the Rihand Reservoir, available for power generation between EL.880.0 and El. 775.0 (Maximum and dead storage levels respectively), is 7.28 million acre feet. Hydrological studies have indicated that 105,000 kW of power could be generated continuously at 100% load factor. In other words, a maximum demand of 200 mW approximately at 50% load factor could be met from this station.

Five units of 50,000 kW each are proposed, to meet a maximum demand of 200 mW with one unit as stand-by. When interlinked with other power systems, Rihand could be utilised for peaking up, water being conserved during the off-peak hours. Being a reservoir-backed station, Rihand could meet a large peak demand, provided sufficient installed capacity is available. Provision for a sixth unit for the future has, therefore, been made in the layout of the power station.

SELECTION OF THE SIZE OF UNITS

The following factors were considered in detail for the selection of the size of the units.

(*a*) Physical dimensions and weights of the packages.
(*b*) The characteristics of power demand.
(*c*) The capacity of the unit at zero leading power factor for meeting charging kVA of the lines.
(*d*) Steady-state and transient stability requirements.

After careful examination of the above factors, 5 units each of 70,000 H.P. corresponding to 50,000 kW or 55,500 kVA at nearly 0.9 p.f. were selected. This selection is in keeping with the modern trend to adopt a fewer number of larger size units, obtaining better economy, efficiency and performance.

TURBINES

(*a*) *Type*: The maximum, minimum and the average gross heads available for the Rihand Power Station Units are 251, 144 and 225 feet respectively. For this range of heads vertical Francis Type turbine is the natural choice.

(*b*) *Rating*: The Turbines have an output of 70,000 H.P. at best efficiency at a net head of 224 feet and restricted gate opening and an output of 70,000 H.P. at 200 feet at full gate opening. The maximum output at heads of 224 feet and higher will be limited to 77,000 H.P.

Turbine Setting and R.P.M.

Preliminary calculation for turbine dimensions and setting indicated that the centre line of the turbine distributor could be set at El. 637. But this has however been set at El. 635.5 to suit the foundation and the tail water conditions at the site (the maxim. and minim. tail-water elevations are 679 and 630 respectively.) This setting permits slightly higher specific speeds than the calculated setting at El. 637 and the corresponding nearest synchronous speed was found to be 150 R.P.M. giving a machine having 20 pairs of poles.

Other Particulars of the Turbines

1. *Runner*: Cast steel, protected by welded-on stainless steel at the outlet.
2. *Shaft*: 30″ diameter of best special Siemens Martin forged steel and bored for the full length.
3. *Swivel gates*: Electrically welded steel, with integral stems.
4. *Speed ring*: Fabricated mild steel, fully stress relieved.
5. *Spiral Casing*: 15′—3$\frac{3}{8}$″ bore at the inlet. Constant velocity type, constructed of mild ness steel plates electrically welded. Maximum thickness=1$\frac{1}{8}$″; minimum thickness=9/16″.
6. *Turbine guide bearing*: Self-lubricated type.
7. *Specific Speed at the design head of* 225 *feet*—45.6 R.P.M.
8. *Maximum Runaway speed*—273 R.P.M.
9. *Normal Speed*—150 R.P.M.
10. *Fly-wheel effect of the turbine*—0.894×10^6 lb. ft^2
11. *Governor sensitivity*—0.01% of speed change.

ALTERNATORS

(*a*) *Type*. The Alternators are of the vertical shaft, salient pole umbrella type with the combined thrust and guide bearing located below the rotor.

(*b*) *Rating*. The rated capacity of the alternators is 55,500 kVA, 11 kV, 50 cycles, 3 phase, and 0.9. p.f.

Salient Features

The alternator stator windings are of the double layer diamond coil type with BSS class 'B' insulation. Closed air ventilation with the coolers tangentially located around the stator frame has been provided. The alternators are equipped with automatic voltage regulators of the control exciter magnetic amplifier

type capable of very fast response. The field poles have laminated cores with damper windings to improve stability and reduce voltage distortion under unbalanced faults. The neutral is grounded through a neutral earthing transformer of suitable capacity and ratio 6.35 kV/110 V with suitable earthing resistance in the secondary. Pivoted pad type, oil lubricated, combined thrust and guide bearings with cooling arrangement for the oil is provided below the rotor.

The generator is protected from fire hazard by the use of nozzles arranged in two groups for initial and delayed discharge of CO_2 around the generator. Two batteries of CO_2 cylinders are used and are arranged for automatic operation from the Generater protection relays (generator differential, generator split phase and generator earth fault). In addition, a switch for manual release of CO_2 is also provided on the turbine guage panel.

Other particulars

1.	Full load efficiency ..	=97.4%
2.	Continuous Overload Capacity at 11 kV, 0.9 p.f. lagging with 80°C stator winding temperature rise ..	61,000 kVA
3.	Output at 11 kV zero p.f. leading with a minimum excitation of 15% of the normal ..	44,000 kVA
4.	Synchronous reactance	
	(*a*) Direct axis	90%
	(*b*) Quadrature axis ..	57%
5.	Direct-Axis transient reactance	
	(*a*) Saturated	32%
	(*b*) Unsaturated	36%
6.	Subtransient Reactance	
	(*a*) Direct axis	23%
	(*b*) Quadrature axis ..	24%
7.	Negative phase sequence reactance	23.5%
8.	Zero phase sequence reactance ..	12.5%
9.	No load balanced telephone influence factor	50
10.	No load residual telephone influence factor	30
11.	Short Circuit Ratio	1.1
12.	Synchronous power—129,000 kW/electrical radian.	
13.	Fly-wheel effect of the rotating parts of the generator and the exciter	41.5×10^6 lb. ft.^{2}
14.	Maximum runaway speed which all parts are guaranteed to withstand	273 R.P.M.
15.	Exciter	286 kW,260V

THE MAIN STEP-UP TRANSFORMERS

The main 11/132 kV step-up transformers are three-phase 60 MVA units and are Delta/Star with the neutral on the 132 kV side solidly earthed, conforming to the Vector Group 42 Yd. 11.

The transformers are forced-oil-water-cooled. Two 100% capacity coolers are provided for each transformer such that one cooler and its equipment can be taken out for repairs or cleaning while the other is in service. A central oil purifier system with suitable size of oil tanks, etc. will be located inside the Power House under the erection bay at El. 650 and the transformers will be connected to these by a suitable piping system.

These transformers will be located on the upstream side between the power house and the dam at El. 683.00. Suitable rail track will be provided for bringing in these transformers to the erection bay under the crane at El. 683.00.

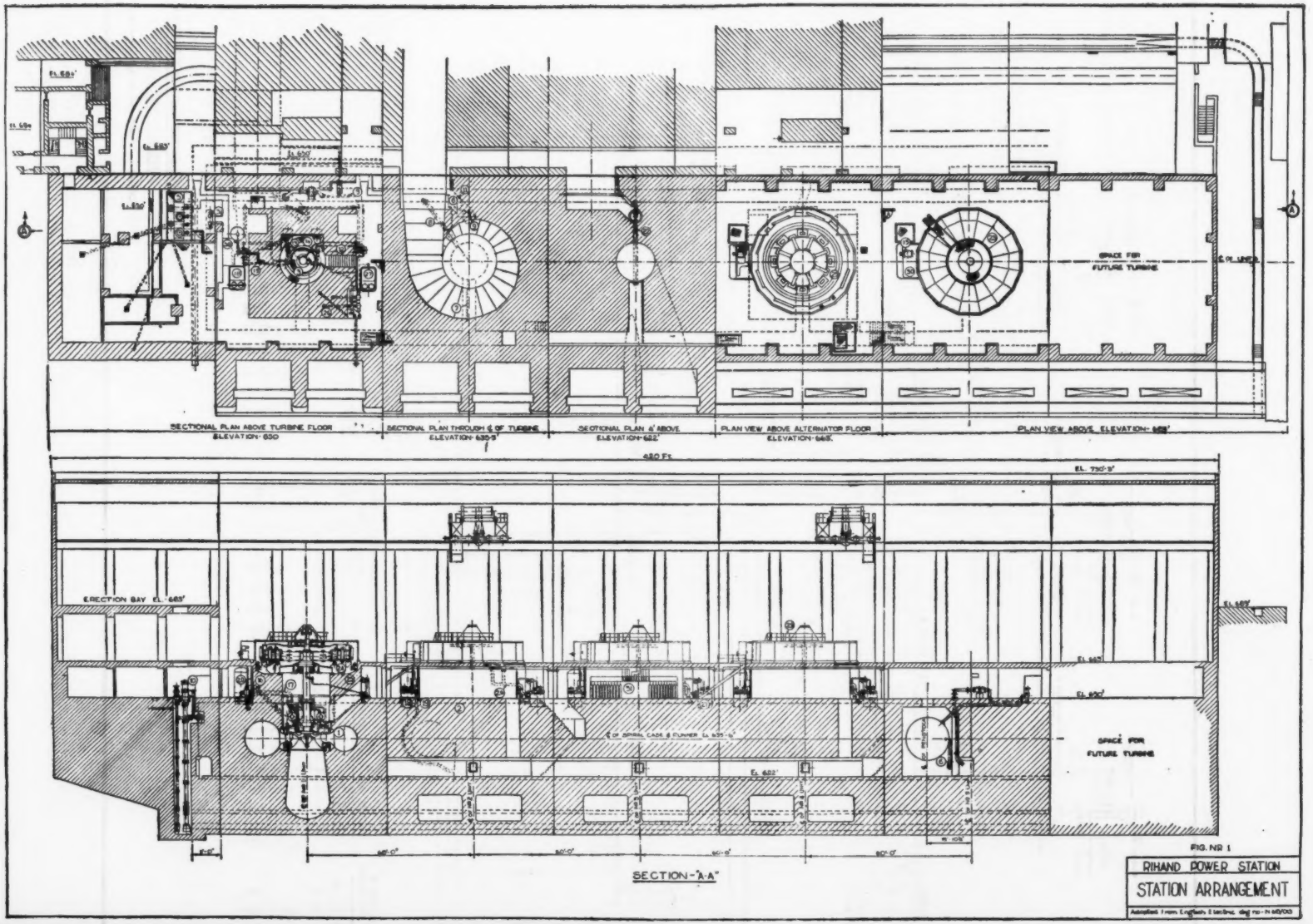
The connection to the 60 MVA transformers from the generators will be through solid copper bars supported on insulators and encased in a light metal sheet cover.

The 132 kV connections from the 60 MVA transformers to the outdoor switchyard will consist of 0.2 sq" C.E., A.C.S.R. conductors anchored to the face of the dam, carried over the roof of the Power House building and terminated at gantry structures in the switchyard.

The transformers are protected by lightning arresters on both 132 kV and 11 kV sides. In case of former the arresters are located on a bracket welded to the transformer tank while in case of latter these are near the generator terminals. Mulsifire System of fire protection combined with baffled walls between each transformer has been provided against fire hazards.

Other Particulars of the 60 MVA Transformers

1.	Insulation on the 132 kV windings	Graded type.
2.	Impulse strength for 1/50 microsec full wave of the 132 kV windings	550 kV (crest) withstand.
3.	132 kV bushing	Condenser type
4.	Tap changing arrangement.	Off load tap-changer with 2.5% and 5.0% positive taps located in the neutral end.
5.	Iron loss at 11 kV primary voltage	62 kW
6.	Magnetising current at 11 kV..	22 amps.
7.	Copper loss at full load unity p.f.	270 kW
8.	Percentage Resistance ..	0.45%
9.	Percentage Reactance ..	10.5%
10.	Percentage Impedance ..	10.5%
11.	Regulation at normal full load and unity p.f. at 11 kV ..	1.0%
12.	-do- at 0.8 p.f. ..	7.0%
13.	Efficiency at U.P.F. loads	
	(*a*) Full load	99.45%



SECTION - "A-A"

FIG. NR 1

RIHAND POWER STATION

STATION ARRANGEMENT

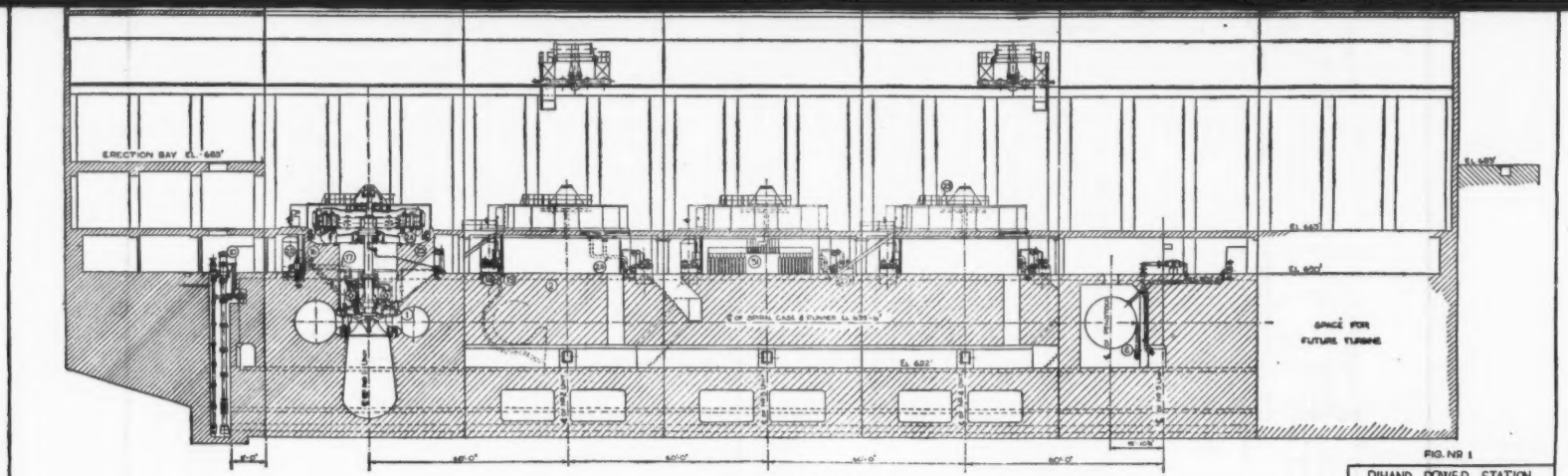
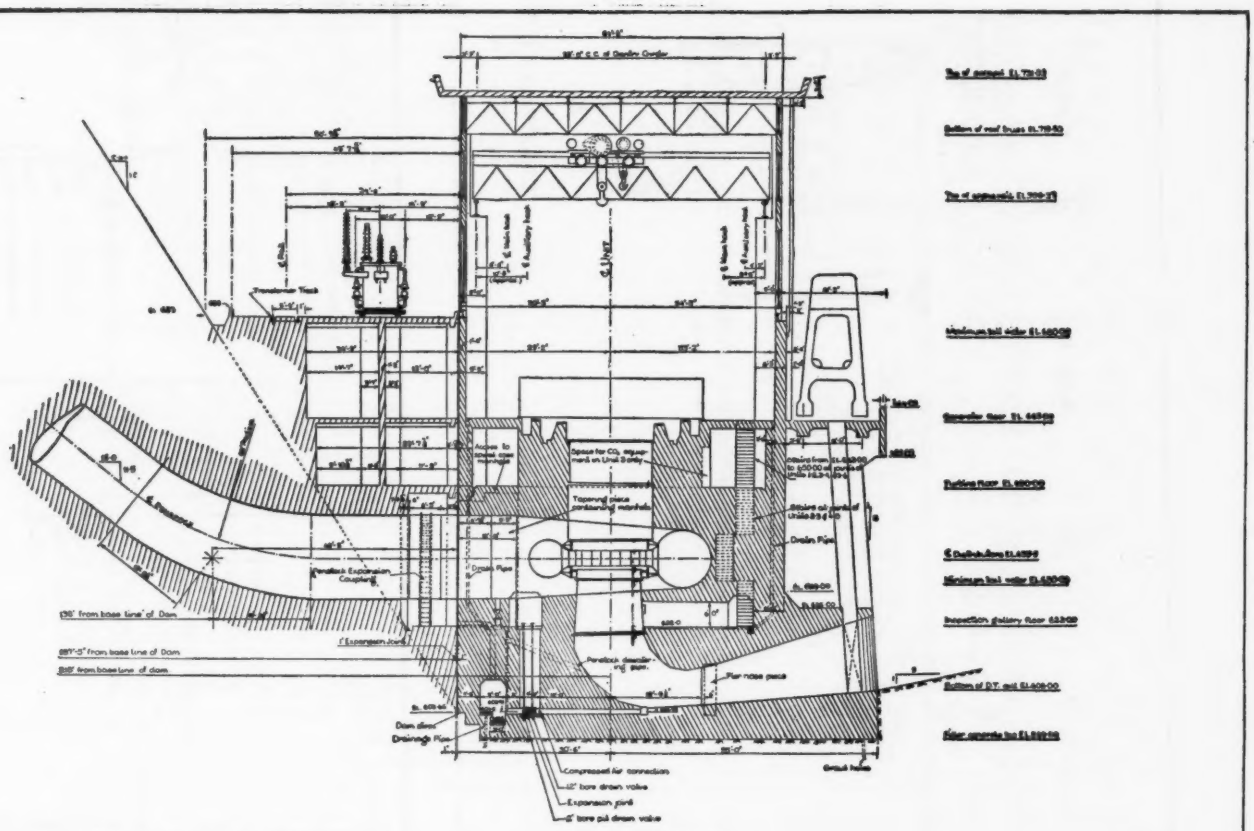


FIG. NR 1

RIHAND POWER STATION

STATION ARRANGEMENT

Accepted from English & Electric drg no - H 60/03


SECTION THROUGH ℄ PENSTOCK
SECTION THROUGH ℄ UNIT
FIG. No.-2
POWER HOUSE
CROSS-SECTION

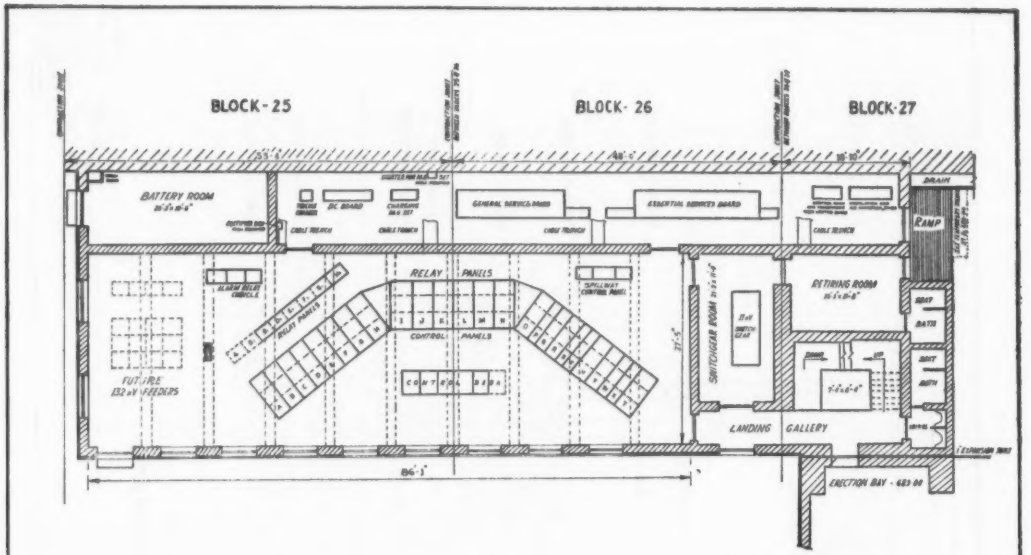
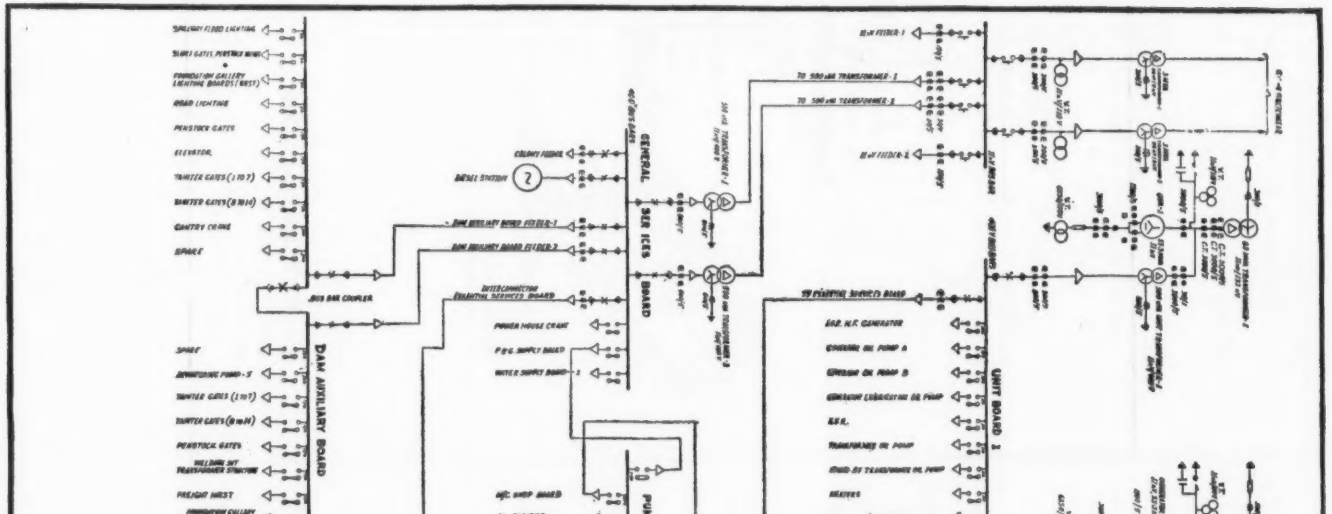
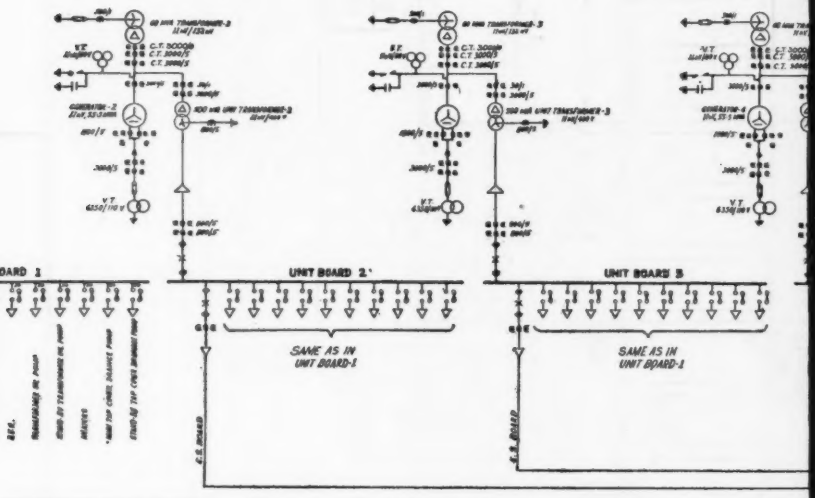


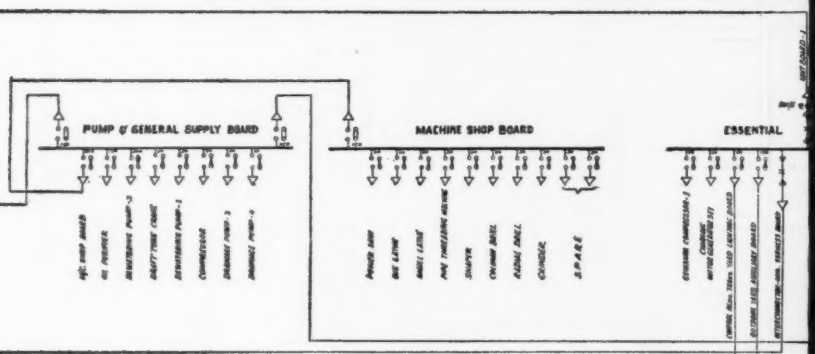
Fig. 3

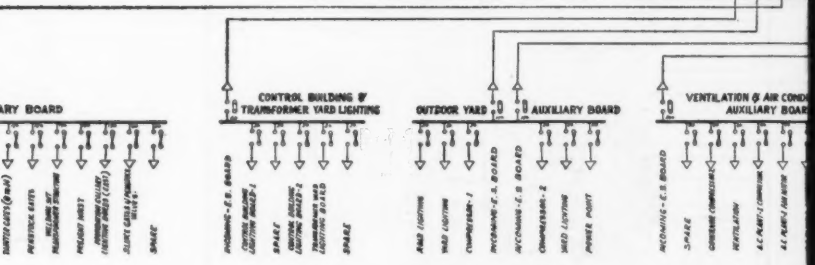
CONTROL ROOM PLAN AT EL 684.00

Adopted from
Design Directorate Drg. No. 3.P.CR.1




UNIT BOARD 2
UNIT BOARD 3
SAME AS IN UNIT BOARD-1
SAME AS IN UNIT BOARD-1
E.S. BOARD
E.S. BOARD


PUMP & GENERAL SUPPLY BOARD
MACHINE SHOP BOARD
ESSENTIAL


CONTROL BUILDING & TRANSFORMER YARD LIGHTING
OUTDOOR YARD
AUXILIARY BOARD
VENTILATION & AIR CONDI
AUXILIARY BOARD

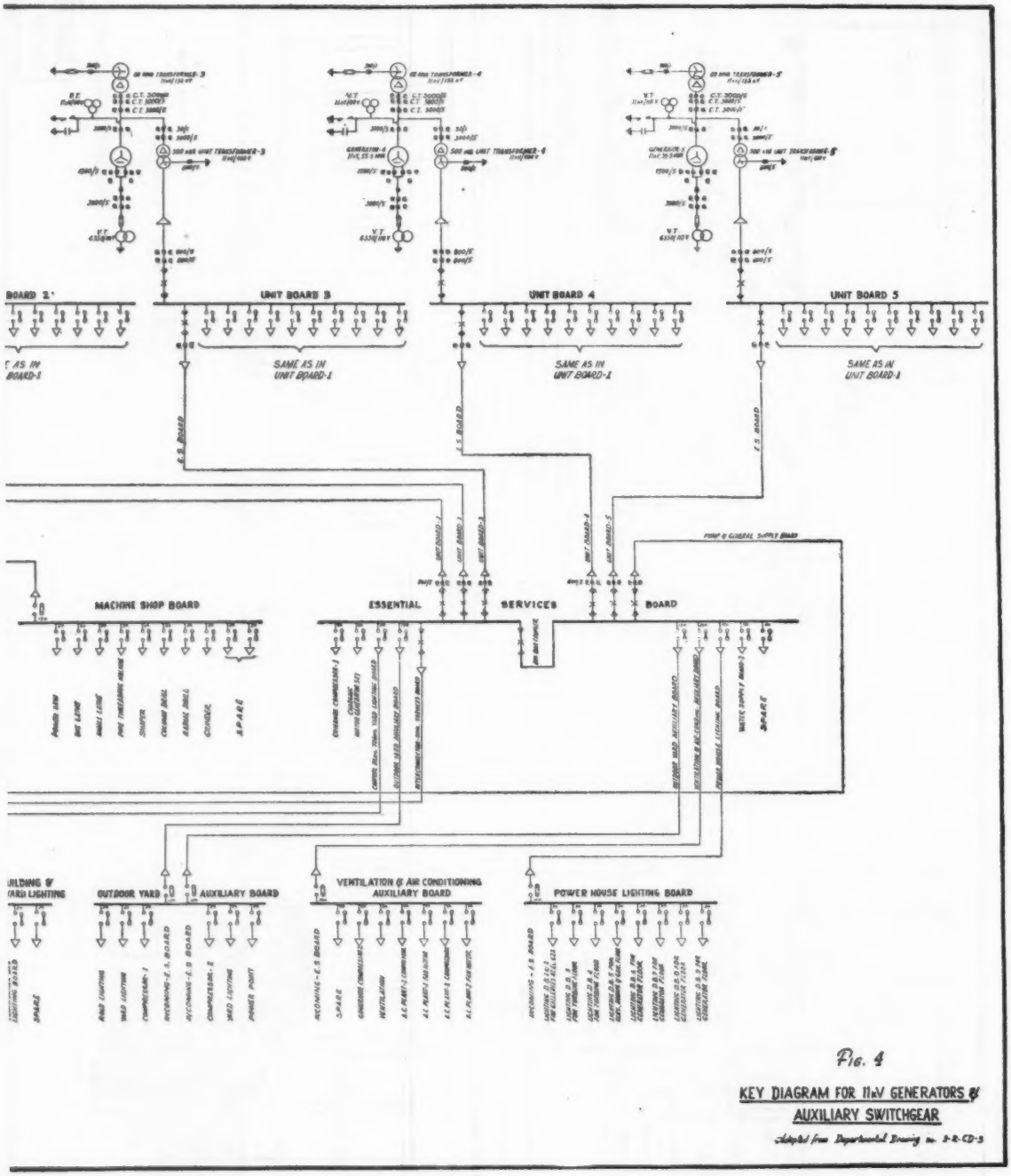


Fig. 4

KEY DIAGRAM FOR 11kV GENERATORS & AUXILIARY SWITCHGEAR

Adapted from Departmental Drawing no. 3-2-CD-3

(*b*) ¾ load 99.52%
(*c*) ½ load 99.57%
(*d*) ¼ load 99.48%

500 kVA UNIT TRANSFORMERS

Each generator will be connected to a 11/0.4 kV, 500 kVA, 3 phase, 50 cycles, outdoor type unit transformer located at El. 683 near the main step up transformer.

The 400 Volts side of these transformers will be connected to the 400 Volt Unit Boards from which the essential generator auxiliaries are fed as also to the Essential Services Board of the Power Station.

500 KVA STATION TRANSFORMERS

Two nos. 11/0.4 kV 500 kVA, 3 phase, 50 cycles outdoor type transformers are also located at El. 683 to provide power supply to the 400 V general Services Board of the Power Station.

These transformers are fed from the switchyard 66 kV bus through two 66/11 kV, 3 mVA transformers.

GENERAL LAYOUT OF THE POWER STATION

The power station is located on the right side of the spill-way approximately opposite dam blocks 27 to 33. It has been divided into seven bays, each of sixty feet width. One of the bays is the erection bay and others are the unit bays. The control room block is a separate building adjacent to dam blocks 23 to 26. The space between the power house building and the dam slope has been utilised to locate the six 11/132 kV 60 mVA step-up transformers, six 11/0.4 kV 500 kVA step down unit transformers, and two 11/0.4 kV 500 kVA step down station transformers at El. 683.00.

The erection bay is at elevation 683 and is approachable by a 30 ft. wide road. A workshop is located below the erection bay at 663.00. Another floor at El. 650 below the erection bay accommodates the oil-storage tanks and the central oil purifier system. Suitable chilling drains have been provided in all the above rooms at El. 650 for disposing off the leakage oil or emulsion of oil and water in case of fire. Sprinkler and mulsifyre type of fire protection system has also been provided in these rooms.

The turbine floor is at El. 650.00. Governor oil pumping sets, alternator lubricating oil pumping sets, pressure receivers, distributing valve, deaeration valves to admit air automatically into the draft tube and the water strainer are all located in the above floor. Cooling water supply pipes with pressure releasing valves etc. are embedded in this floor. Stairs for getting down to the inspection gallery at El. 622 from the El. 650 floor are provided at two places viz., between units 2 and 3 and units 4 and 5. Also individual stairs to the turbine pits are provided from the 650 floor.

A cable gallery to the control room block is also situated at El. 650 on the upstream of the power station.

The generators are located at El. 663. The turbine actuator and instrument panel is located near each unit. The automatic voltage regulator boards and the unit boards are located in the gallery at El. 663. The pump and general services board is also located in this gallery. Space for untanking the 60 MVA transformers is provided on the Generator floor in unit bay no. 1.

A gallery for the drainage of various floors, the turbine top covers and dewatering of the draft tubes has been provided on the upstream side of the units, at El. 603.75. The gallery terminates into a dewatering sump with bottom at El. 599.5 located in the erection bay block. The bottom of the draft tube of each unit is connected to the drainage gallery through a 12" pipe with a valve operated from the draft tube inspection gallery at El. 622. The water leaking through the runner-seals of the turbines is pumped automatically by two pumps controlled by electrodes, to the drainage gallery. Various floors are drained to the drainage gallery by steel pipes embedded at suitable locations.

The water collected in the dewatering pit is pumped out into the tail race by means of 4 nos. electrically controlled automatic starting bore hole pumps. Two of these pumps are of 300 gallons per minute capacity and are meant for the normal drainage of the power house, while the other two pumps are of 3000 gallons per minute capacity and are meant for dewatering of the draft tubes. In addition to these, a pump of 3000 gallons/min. capacity is provided in the spillway training wall with its motor at El. 683. This provision has been made to enable dewatering in the unlikely event of the power house getting flooded.

The layout of control room block situated adjacent to the power station building overlooking the outdoor switchyard comprises briefly as below:—

(*a*) Cable basement at El. 673: Cables run into this floor from the cable gallery of the power station, at El. 650 and the outdoor switchyard at El. 683.

(*b*) Control room, 11 kV switchgear room, D.C. room, and Essential services boards at El. 684.

(*c*) Office rooms, Committee room, telecommunications room, air conditioning equipment room, etc. at El. 699.63.

A control and relay board for each generator and feeder is provided in the control room. This board is of sheet steel corridor type box construction with control switches and instruments in the front and relays on the rear. A separate control desk alongside the table of the operater is provided for operation of each machine.

The power station is equipped with two 90 ton electrically operated overhead travelling cranes ope-

rated from 400 V, 50 cycles, 3 phase supply. A lifting beam designed to lift a load of 180 tons by the two cranes is also provided.

The layout of the outdoor switchgear and scheme of protection for the station are described in details in other articles.

LIGHTING OF THE POWER STATION AND THE CONTROL ROOM

The lighting scheme of the various portions of the power station and the control room block is a compromise of utility.

The generator hall is 420′ long 62′ wide and the height of the roof truss from working floor is 57′. 400 W, high pressure mercury vapour lamps blended with 750 W incandescent lamps have been selected to give an illumination level of 7 lumens per sq. ft. The lamps shall be fitted in combined lighting units which shall also provide good upward illumination. In all, there shall be 28 such units.

Fluorescent lighting shall be provided in workshop at El. 663 and El. 650. The fitting shall consist of industrial trough type reflectors suitable for accomodating 2×40 W fluoresceent tubes. The level of illumination in the turbine floor is 5—7 lumens per sq. ft. and that in the workshop 10 lumens per sq. ft. Good number of socket out-lets at suitable points have been provided for any additional local lighting required during inspection and overhaul of the equipments.

A considerably high level of illumination of 20 lumens per sq. ft. has been decided for the control room as it is the nerve centre of all the operations and good illumination is an absolute necessity for easy reading of the measuring devices and the manipulation of the control knobs.

For the offices and the committee room 2×40 watt flourescent tubes in decorative fittings are provided, the average illumination given being 10 lumens per sq. ft.

The switchyard, transformer-bay, the Power House exterior and the control room block exterior are flood lit with incandescent lamps. The approach road is to be lighted by 4×20 W—2 ft. fluorescent tube in post top lantern fittings. The staggered lighting arrangement has been adopted.

AIR CONDITIONING

For the efficient working of delicate instruments installed in the control and switchgear rooms and operating staff control room, switchgear room and the main office rooms have been air conditioned. Two 12.5 tons packaged air conditioning units shall be installed on the first floor of the control building, for this purpose.

In the end, I must thank my colleagues in the design office, who have helped in preparation of this article.

(Continued from page 98)

SCHEME OF PROTECTION AT RIHAND POWER STATION

(iii) Damping elements for unwanted oscillations set up in the circuit and to prevent occurrence of D.C. component has been added.

With all these features the relay is capable of securing selective tripping in the event of single pole and triple pole short circuits.

Phase Selection Relays

The relay L6f is also provided with phase selection relay of product type. The product relay ensures correct phase selection for faults with heavy infeed (with small voltage drop at the end of a long line where because of low current the impedance relay cannot respond for similar faults with minimum infeed.). The relay is supplied with the product of two voltage and responds immediately when the product falls below the set value or becomes negative.

Carrier Relaying

For simultaneous tripping of breakers at both ends under internal fault condition, use of carrier has been made. The circuit in the protective relay is so arranged that for internal faults, the relay unit which operates in first zone sends a tripping impulse to other end. Therefore for carrier relaying two phase or intersystem coupling is necessary. The frequency shift carrier protection equipments (fixed frequency equipment) working in conjunction with carrier telephone cabinets within the telephone band itself have been provided for the Rihand Grid. During the short operation time of inband frequency shift carrier protection equipment the telephone circuits will temporarily be switched off to provide full power for sure operation of protection equipment.

The speed of operation of protection equipment is 15 milli seconds.

I gratefully acknowledge the help of my colleagues in Design Office in preparation of this article and also of the pamphlets and drawings of Messrs. Brown Boveri and Messrs. English Electric Co., the suppliers of Protective Gear and Power Plant for the Rihand Power Station respectively.

Earthing Mat and Voltage Gradients Study

By K. R. AGARWALA

THE site of Rihand Power Station lies in rocky ground having high resistivity. Special attention was, therefore, needed in the design of grounding system to protect life and equipment. A reliable and low resistivity earth is also required for proper functioning of protective devices and draining high voltage surges. It is also necessary to earth all metallic non-current carrying parts of the equipments such as machine frames, metal hand rails, cabinets and conduits, tanks of Power transformers, framework of oil circuit breakers and steel structures etc.

The grounding system has to be such, that the potential of ground in the neighbourhood of the earth electrodes and the voltage gradient around them do not endanger a person stepping on that ground or damage the buried telephone and protective gear cables.

It has also to be ensured that the voltage difference between the earthing bar and ground is within limits, so that a person standing on the ground and touching the bar is not injured or electrocuted.

It is necessary therefore to provide a low resistance grounding installation dependable throughout the year.

According to present day practice the maximum value of resistance of grounding electrode for the generating station will be kept within 0.5 ohm and for transformer station (over 100 kV) 1.0 ohm.

TYPE OF GROUNDING MATS

The area upsteam and downstream of Rihand Dam is such that rock is present very near the surface and as such the only solution was to have a buried grid of copper conductors embedded in the ground about 2 ft. below the surface.

In the switchyard area the rock is present at greater depth and as such it is possible to have a pattern of copper or galvanised iron rods driven into the ground along the boundary of the switchyard with their tops interconnected with copper conductors.

FACTORS AFFECTING DESIGN OF MATS

The resistance of mats with regard to the ground depends upon the type of material in which the mat is laid. It varies considerably with the moisture contents of the soil. The greater the moisture content the lower is the resistivity of the soil. A reduction in temperature in general increases the resistivity, with a pronounced rise when freezing occurs.

The resistivity at Rihand Dam site was measured by a four-electrode "Megger" and the following average values were obtained:—

Upstream	.. ..	1000 ohms/cm^3
Switchyard	.. ..	5000 ohms/cm^3

The change in moisture content and temperature for upstream mat will be almost negligible since this mat will always be under water. The design value of resistivity of soil for upstream mat has been adopted at a safe figure of 5000 ohms/cm^3. The same value has been used for a mat in the tail race.

The variation of moisture in the earth surrounding the switchyard mat may be quite appreciable. This mat, however, is accessible and artificial treatment can be done to ensure moist conditions throughout the year. A figure of 10,000 ohms/cm^3 has been used in the calculations in this case.

Dimensions of the mat and spacing of cross conductors have to be fixed keeping in view the following points:—

(i) Simple loops encirling the area over which the mat is to be laid are most economical.

(ii) Cross connections do not materially lower the resistance of the grid but are desirable because of mechanical considerations.

(iii) Substantial savings can be made by spreading the grid over larger areas, while very little is gained by increasing the number of meshes.

(iv) Selection of materials and cross sectional dimensions of the mat conductors are primarily governed by mechanical strength rather than considerations of corrosion and current capacity.

(v) Substantial advantage can be gained by arranging the grid of a given area in a narrow strip. As the length/width ratio of the mat is increased, the change in the ground resistance of mat with respect to variation in depth becomes less pronounced and for depth to be used for a grounding mat this change in resistance may be neglected if the ratio is greater than 4.

(vi) The layout of the mats has to be such that the fault currents do not produce hazardous ground potentials and voltage gradients for the operating personnel.

In addition to the above the following features have

been adopted in the design of grounding mats:—

(i) The minimum size of conductors are 4/0 A.W.G. bare stranded copper.
(ii) The outside ring of the ground mat has the same copper section as the risers.
(iii) Conductors are spaced approximately 40-50 ft. apart in both directions in the ground mat.
(iv) All cable crossings are clamped together with bronze or stainless steel ground clamps or carefully brazed.
(v) At least two cable risers are taken from each mat to the test stations for connection to various equipments.
(vi) Two separate earthing connections are provided for each equipment in accordance with Indian Electricity rules 1956.

It has been found necessary to have a mat in the upstream and another in the tail race and a system of grounding rods for the switchyard.

DESIGN OF UPSTREAM GROUNDING MAT

The ratio of length to width of the mat has been kept equal to 4. A mat of 500 Mcm. (19/0.166 S.W.G.), having overall diameter of 0.811 inch, copper conductors of 100 ft.$\times$400 ft. outer dimensions, the intermediate conductors being spaced at 50 feet interval in both directions has been proposed.

This mat is to be buried 2 ft. deep in the ground either by excavating trenches and back filling these or by laying conductors on the ground and covering these with sandy clay.

The mat is illustrated in figure 1.

The resistance of the mat was calculated by various methods which are mentioned below:—

1. *Method of A.H. Mc Cracklin and C.W. Wend Landt* ("Determination of Resistance to ground of grounding grids" Transactions A.I.E.E. 1952 part III volume 71).

The authors have actually measured the resistance of grids for various length/breadth ratios and placed at various depths. The results of these tests have been presented in the form of curves and the authors have given an empirical equation for the resistance of the mat.

2. *Method of S. J. Schwarz* ("How to determine ground resistance of horizontal grid and multiple rod system") Electrical World, May 2, 1955 and also A.I.E.E. Transactions 1954 vol. 73 part III.

The author has given an emperical equation in terms of constants which are determined from curves given by the author.

3. *Method of Gross and Wise* ("Grounding grids for high Voltage stations"—Resistance of large rectangular plates")

By this method the value of resistance to ground of a plate of corresponding size for calculating the resistance of a rectangular mat is determined since the plate is the limiting case of any mesh system, and a plate would give the minimum resistance possible.

The resistance of the upstream mat worked out by the first two methods is 0.4 ohms. The limiting value of the resistance of grid is 0.328 ohms as worked out by the third method. All these values are less than the maximum permissible value of 0.5 ohms and as such the size of the mat is adopted.

DESIGN OF SWITCHYARD MAT

A mat 200'$\times$360' consisting of 56 No. 3/4 inch diameter rods each 10 ft. long spaced at 20 feet centre to centre and connected by 500 MCM wire is considered. The mat is illustrated in fig. 1.

The resistance of this mat was calculated by various methods taking the value of soil resistivity as 10,000 ohms/cm^3. The various methods are mentioned below.

1. *Method of S. J. Schwarz*:—("How to determine ground resistance of horizontal grid and multiple rod system")

   Electrical world, May 2, 1955 and also A.I.E.E. Transactions 1954 vol 73 part III.

   An empirical equation is given by the author in terms of constants which are determined from curves given by author.

2. *Method as given in "Standard Handbook for Electrical Engineers"* by A. E. Knowlton 9th edition 1957—Sec. 13, page 1293.

In this method the conductivity of a single ground rod is calculated by means of conductivity ratio curves given by the author. Knowing the conductivity of the rod grid, its resistance is calculated.

The value of resistance of switchyard grounding electrode as worked by the above two methods is very nearly equal to 1 ohm and as such the mat has been adopted. Necessary modification in shape according to requirements of layout of switchyard will have to be made.

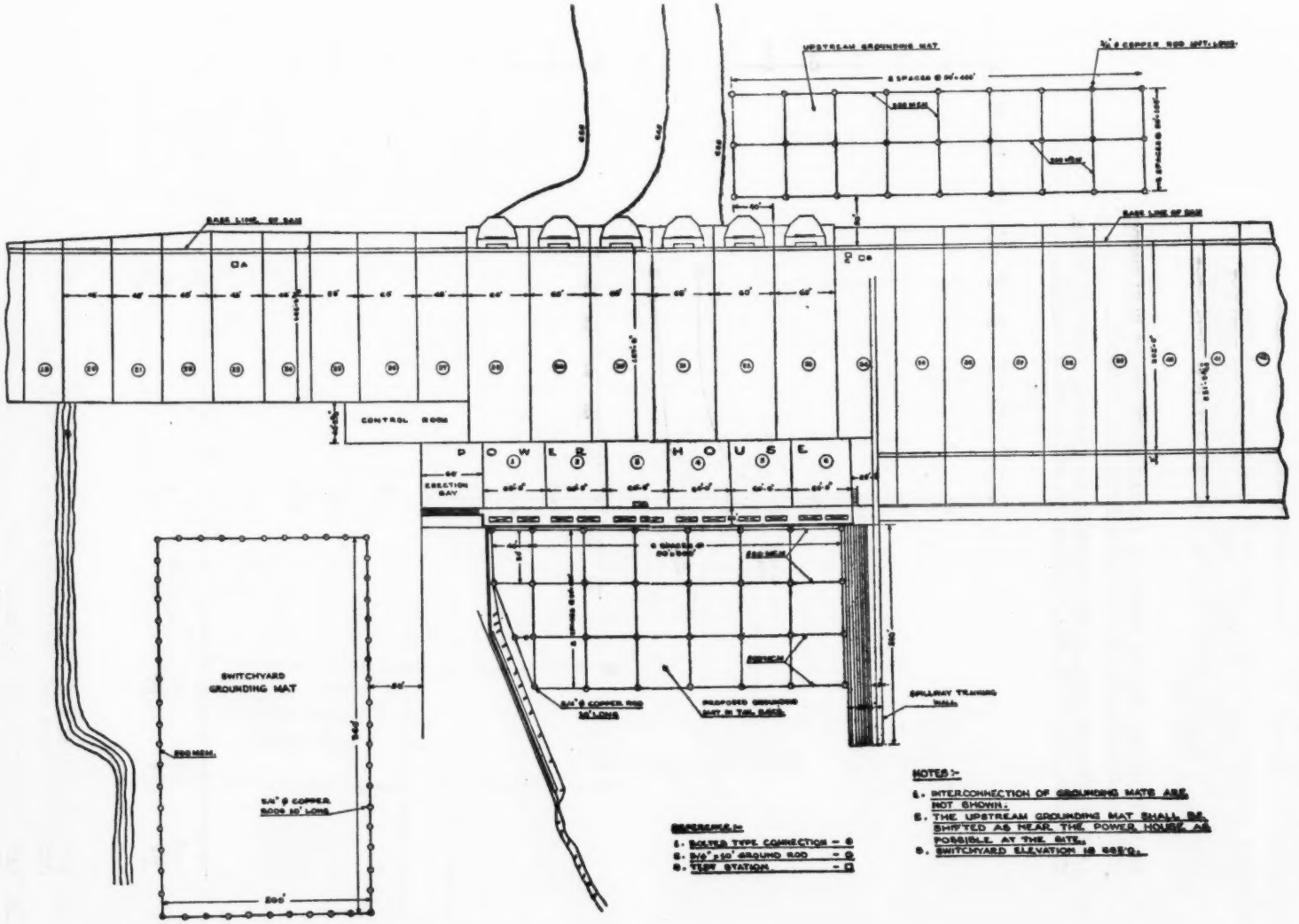
DESIGN OF TAIL RACE MAT

The mat is illustrated in fig. 1 along with other mats and its resistance was worked out in accordance with methods as for upstream mat.

The resistance of the tail race grounding mat also works to 0.4 ohms approximately, which is within required limits.

COMBINED RESISTANCE OF VARIOUS MATS

The combined resistance of upstream and switchyard mats is computed from their individual resistances as determined above and from an equation given by S.J. Schwarz ("How to determine resistance of horizontal grid and multiple rod systems") Electrical World, May 2, 1955.

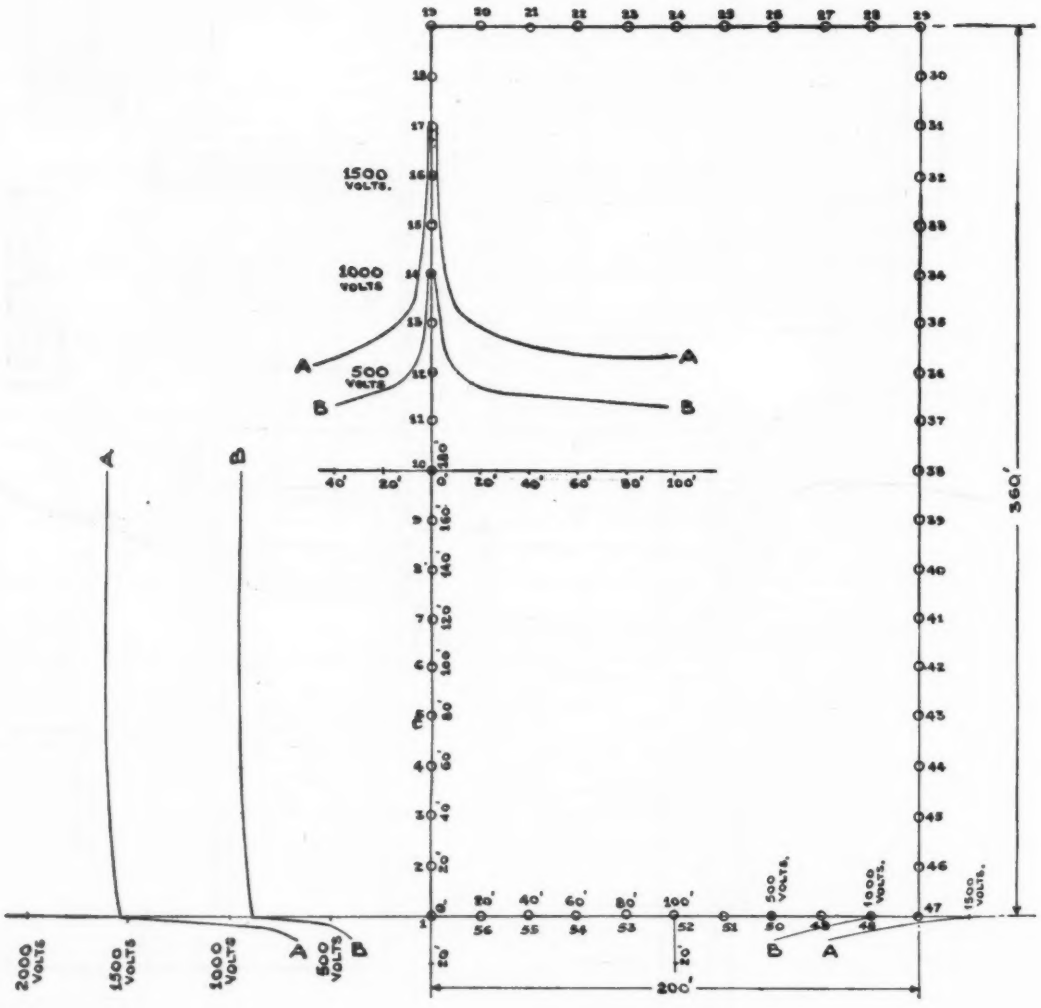

UPSTREAM GROUNDING MAT
BASE LINE OF DAM
CONTROL ROOM
POWER HOUSE
ERECTION BAY
SPILLWAY TRAINING WALL
SWITCHYARD GROUNDING MAT
200 MESH.
200'
PROPOSED GROUNDING MAT IN THE BASE.
NOTES :-
1. INTERCONNECTION OF GROUNDING MATS ARE NOT SHOWN.
2. THE UPSTREAM GROUNDING MAT SHALL BE SHIFTED AS NEAR THE POWER HOUSE AS POSSIBLE AT THE SITE.
3. SWITCHYARD ELEVATION IS 695'0".

1. WHEN UPSTREAM AND SWITCHYARD MATS ONLY ARE PRESENT. MAXIMUM POTENTIAL IS AT POINT (0,180FT.) = 1658·7 (DUE TO CURRENT THROUGH SWITCHYARD ELECTRODE) + 136 (DUE TO CURRENT THROUGH UPSTREAM ELECTRODE) = 1794·7 VOLTS.

2. WHEN UPSTREAM, TAIL RACE AND SWITCHYARD MATS ARE PRESENT. MAXIMUM POTENTIAL IS AT POINT (0,180 FT.) = 976 (DUE TO CURRENT THROUGH SWITCHYARD ELECTRODE) + 80 (DUE TO CURRENT THROUGH UPSTREAM ELECTRODE) + 204 (DUE TO CURRENT THROUGH TAIL RACE ELECTRODE) = 1260 VOLTS.

"A" EARTH POTENTIAL DUE TO CURRENT THROUGH SWITCHYARD ELECTRODE WHEN UPSTREAM AND SWITCHYARDS MATS ARE PRESENT.

"B" EARTH POTENTIAL DUE TO SWITCHYARD ELECTRODE CURRENT WHEN UPSTREAM TAIL RACE AND SWITCHYARD MATS ARE PRESENT.



"A" EARTH POTENTIAL DUE TO CURRENT THROUGH SWITCHYARD ELECTRODE WHEN UPSTREAM AND SWITCHYARDS MATS ARE PRESENT.

"B" EARTH POTENTIAL DUE TO SWITCHYARD ELECTRODE CURRENT WHEN UPSTREAM TAIL RACE AND SWITCHYARD MATS ARE PRESENT.

FIG. 2

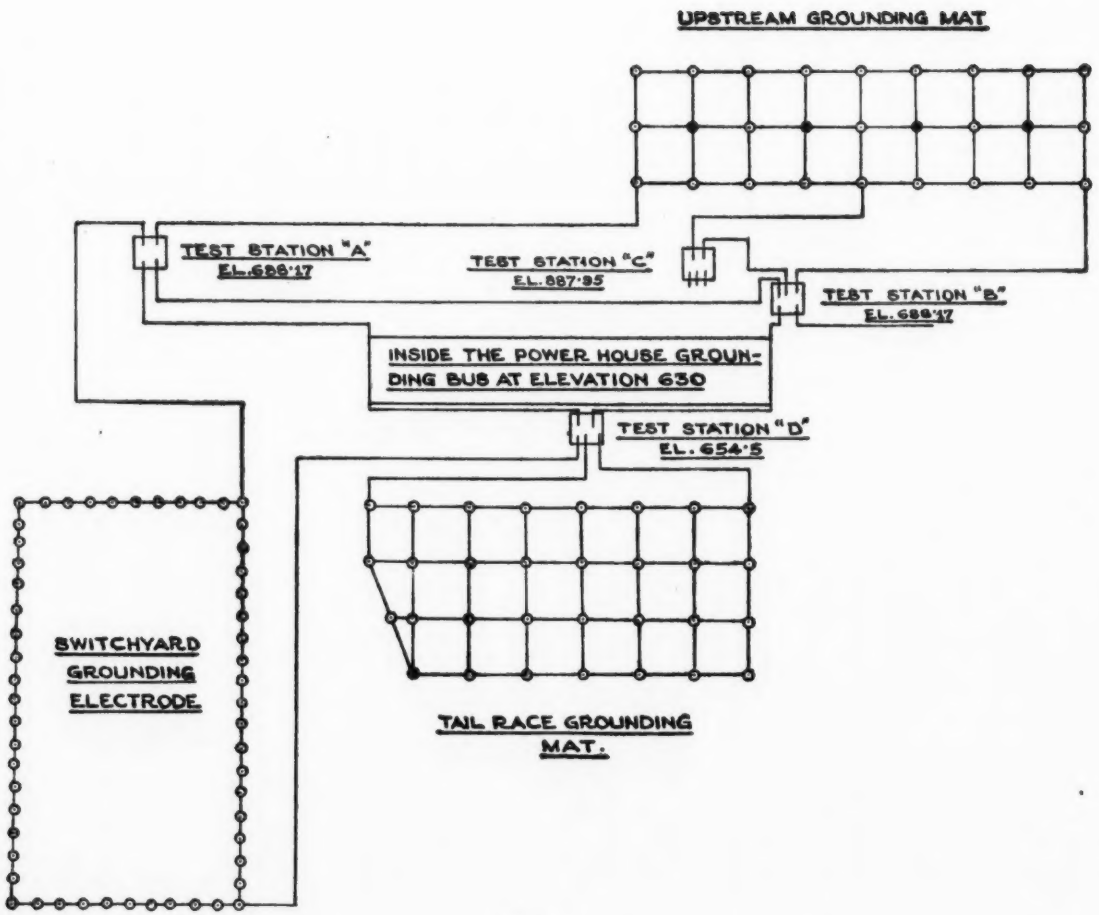

UPSTREAM GROUNDING MAT
TEST STATION "A"
EL. 688'17
TEST STATION "C"
EL. 887·95
TEST STATION "B"
EL. 688'17
INSIDE THE POWER HOUSE GROUNDING BUS AT ELEVATION 630
TEST STATION "D"
EL. 654'5
SWITCHYARD GROUNDING ELECTRODE
TAIL RACE GROUNDING MAT.

FIG. 3

The soil resistivity in this case is taken as the mean of resistivity in upstream of dam and switchyard i.e. 7500 ohms/cm³ or 75 ohm-meters.

To determine the combined resistance of all three earth electrodes the combined resistance of upstream and tail race mats is first found assuming as if the mats were placed side by side i.e. neglecting the resistance of interconnectors by the method of S.J. Schwarz and the resistance thus found and that of the switchyard electrode is combined by the equation given by the same author.

The combined resistance of upstream and switchyard electrodes works out to 0.4 ohms and of all the three electrodes 0.283 ohms.

FAULT CURRENTS

Having determined the resistance of the various mats and their combined resistances it is necessary to find out the maximum fault currents for the study of voltage gradients. The fault currents are determined for two cases (1) The upstream mat and switchyard electrodes are present (2) Both upstream and tail race mats and switchyard electrodes are present.

There are 6 Nos. 55, 500 k.V.A. 11 kV generators each feeding a 60 MVA, 11/132 kV Delta/Star transformer arranged in the unit scheme. There is a common bus on the 132 kV side.

The design of the neutral grounding transformer for generators is such that the fault current with a terminal earth fault on the generator at 11 kV does not exceed 2913 amps., the full load current of the generator. The maximum fault current occurs on a phase to earth fault on 132 kV. busbar.

The subtransient reactances of the generators are considered since these give the maximum fault currents and thus cause maximum voltage gradients.

The fault current when only upstream and switchyards mat are present works out to 5680 amperes.

The fault current when all the three mats are present works out to 5727 amperes.

POTENTIAL GRADIENTS

General

The following points in respect of voltages developed during earth faults have been considered:—

1. Potential to which the ground in the switchyard is raised.
2. The nature of voltage gradients developed.
3. The effect of touching of an earthing bar by an operator by chance.

The effect of current flow through an earth electrode is that a voltage gradient is present on the surface of the ground in the immediate neighbourhood of the electrode. The fundamental reason for this gradient is that the resistance of the electrode is not concentrated at one point but is distributed over the soil in its vicinity. A considerable voltage can exist over a relatively short distance and if, for example, a four footed animal stands with the centre line of his body radial to the electrode, a potential may exist between his fore and hind legs possibly resulting in a fatality. Similarly a human being taking stride in the neighbourhood of a current carrying electrode can also be affected.

The danger due to a voltage gradient around current carrying electrode is greatest with single electrode systems. By using two or more electrodes spread over the station, this effect is largely eliminated due to the increase in area and parallel paths for the conduction of fault current.

The simplest possible electrode is a sphere in the ground which is symmetrical in all directions with the lower hemisphere buried in the ground.

If a current I flows through this electrode spreading out radially in the ground, the current density at distance X from the centre of the hemisphere is $i = \frac{I}{2\pi \times X^2}$ and the electric field strength, as voltage per unit length is $e = \frac{P\,I}{2\pi X^2}$, where P is the resistivity of the soil in ohms/cm³.

If a person is walking through such an electrical field the body diverts some current from the earth and may suffer damage from potential gradients. The field strength produces a current in any living creature stepping across it. The maximum possible current develops if the internal body resistance is small compared with the foot resistance on the ground which is $r = \frac{P}{2\pi b}$ where b signifies the equivalent radius of the creature's foot. For a man's foot this has been measured as b=7 cm. on the average. Since the voltage drop by the body current through the resistance 2r due to both feet is given by the voltage taken over the step length 's' is $2ri = e_s$ and the maximum body current becomes $i_s = \frac{e_s}{2r} = \frac{\pi\, b\, e_s}{P}$

The minimum fibrillation current for human beings is in the region of 0.1 ampere at 50 cycles.

Hence $0.1 = \frac{\pi}{P} b\, e_s$

$\therefore\ e_s = \frac{0.1\,P}{\pi b} = \frac{0.1 \times 10000}{3.14 \times 7} = 46$ volts,

Hence the voltage difference between the two feet of a man should not be more than 46 volts. This figure is based on an earth resistivity of 10,000 ohms/cm³ as decided before for the mat under the switchyard.

Another effect of the earth fault current is to raise the potential of ground above the zero earth potential. In case this potential is allowed to go pretty high there is a danger to control cables and other apparatus in the vicinity.

CALCULATION OF EARTH POTENTIALS

For a vertical ground rod or pipe of length L, the

earth potential at the surface at the distance Y from the rod is obtained by the formula,

$$U(Y)=\frac{IP}{2\pi L}\,\text{Log}_e\,\frac{(L^2+Y^2)^{\frac{1}{2}}+L}{Y}$$

The earth potential can also be obtained in the vicinity of a buried horizontal wire. For a wire of length L buried at a depth d, the potential at a point on the surface at the horizontal separation Y from the wire and a longitudinal distance x from a line through the midpoint of the wire is given by the formula:

$$U(x,y)=\frac{IP}{2\pi L}\,\log_e\,\frac{\left\{(x-\frac{L}{2})^2+y^2+d^2\right\}^{\frac{1}{2}}+x+\frac{L}{2}}{\left\{(x-\frac{L}{2})^2+y^2+d^2\right\}^{\frac{1}{2}}+x-\frac{L}{2}}$$

When there are a number of rods or wire, the resultant earth potential is obtained as the sum of earth potentials due to each, the current in each of n equal rods or wires being taken as $\frac{I}{n}$.

"Earth conduction Effects in Transmission system" by Erling D. Sunde.

The steepest voltage gradients and maximum potential rise will be caused when the individual earth connection of an apparatus is disconnected and the whole fault current phases through the mass of earth to the electrodes from the point of fault on an equipment inside the substation. The voltage gradients in the power house will not be so steep since it it lies away from the resistance area of the mats.

The possibility of a dead earth fault is greatest in the switchyard and the maximum rise of potential will as such be in the switchyard in the vicinity of electrodes because the effective resistance area is concentrated within about 10′ to 15′ of the electrodes.

For purposes of calculation of potentials it is assumed that the total fault current divides itself between the various electrodes in the ratio of their conductances approximately.

The voltages at various points in the switchyard have been worked out. The curves 'A' (See fig. 2) have been drawn for the potential effect, due to current through switchyard electrode at various points on the switchyard when there are two electrodes i.e. upstream mat and switchyard electrode. It is seen that the potential is almost constant (of the order of 1600 volts) along longitudinal boundary of the switchyard nearer to the power house. At the corners of the switchyard and at short distances from boundary there are steep potential gradients.

The effect of the portion of fault current passing through the upstream mat is found to raise the overall potential of switchyard by 136 volts approximately throughout.

The maximum potential at the switchyard in this case works out to 1795 volts approximately.

The curves 'B' (see fig. 2) have been drawn for potential effect due to current through switchyard electrode at various points on the switchyard when the tail race mat is also present. The overall potentials and gradients are reduced in this case. The maximum potential on the switchyard in this case works out to 1260 volts taking into account 80 volts as the effect of upstream mat and 204 volts that of tail race mat as calculated.

From the studies as indicated above it has been found desirable to use a grounding mat in the tail race as well to reduce the overall ground potentials and voltage gradients. The provision of a mat in the tail race is necessary from another point of view also. During testing when the upstream grounding mat will be isolated from the grounding system there will be no other equally effective electrode and if an earth fault occurs during that period the equipments and operators are likely to be put to unwarranted hazards.

The is also the danger to a person standing on the substation floor and touching earthed metallic parts of the plant or the earthing strips. This requires that any surface on which a person may be standing is substantially at the same potential as surrounding objects.

The potential of ground which is minimum in the centre of switchyard is calculated to be 557.2 volts for a case when all the three electrodes are in service.

The potential of earthed metal parts of the plant of earth strip under fault on the system will be the combined resistance of the three electrodes multiplied by the total fault current that is $0.283\times5727=1620.7$ volts. Thus a voltage of 1620.7—557.2= 1063.5 volts will act on a person standing in the centre of switchyard and touching any object such as metallic frame of the equipment or earth strip.

Assuming a resistance of 8000 ohms for a (man's) body the current through the body when tail race mat is also present will be $\frac{1063.5}{8000}=0.133$ ampere. This current is in excess of the limiting value of 1/10 ampere.

However, the above hazard will be substantially removed since buried grids well connected to surrounding structures, circuit breakers, isolators and the like have to be necessarily arranged for earthing the individual equipments.

GROUNDING CONDUCTORS AND METHODS

Ground terminal boxes have been provided at low accessible elevation in the power House. These are also used as test points to determine the ground resistance. Fibre conduits are to be used at the entrance of risers to minimise unwanted short circuits between mats.

The following precautions are to be taken in the installation of grounding system:—

A. A ground cable is not to be laid close to or parallel with generator main leads or other single phase power leads.

B. A ground cable is not to be run through a conduit of magnetic material or through a hole or other opening in magnetic material.

C. The lead sheath of a power cable is not to be grounded at more than one point.

D. A current transformer circuit is not to be grounded at more than one point.

It is proposed to have brazed connections in Rihand Dam grounding system for risers from test stations and connections to various parts of plant and clamp type connections for grounding mats and test stations. The grounding mats have been suitably anchored in the upstream and tail race.

Lightning conductors will be provided on top of passenger and freight elevator towers. General practice in case of lightning conductor is not to adopt a conductor weighing less than 3 ounces per 1 linear foot and accordingly 4/0 A.W.G. size conductors have been adopted.

Ground terminal boxes or test stations have been provided at four locations, these being:—

A. Penstock operating gallery opposite switchyard adit.

B. Penstock operating gallery opposite power house adit.

C. Intake gate hoist operating gallery near passenger elevator end.

D. Opposite centre of 3rd. machine on the inside of tail race side of wall on turbine floor.

These stations have connecting links which permit isolation of different parts of the grounding system and measurement of resistance of each part.

The system of main leads from grounding mats to test stations, risers distributed throughout the Dam and connections to individual items of plant is laid out as described hereafter.

MAIN LEADS

Three main leads of 500 MCM(19/0.166 S.W.G.) conductor each will be taken inside the dam from the upstream ground mat, these being connected one each to test stations A, B and C.

Two main leads of 500 MCM conductor will be brought from switchyard grounding electrode system one connected to the upstream mat through test station A and the other to tail race mat through test station D.

Two main leads each of 500 MCM conductor will be brought from the tail race mat to test station D.

The power house ground bus system will be connected by 500 MCM conductor by one lead each to test station A and B on one side and by two leads to Test Station D on the opposite side. A similar lead almost vertically through the dam will connect test station C with test station B. Another similar lead running through the penstock operating gallery will connect stations A and B.

All risers from test stations will generally be of No. 4/0 A.W.G. or 37/.083 S.W.G. conductor except for elevator machinery which will be 2/0 A.W.G. or 19/.083 S.W.G.

Connections to individual machinery will be done by a minimum No. 4 A.W.G. or No. 5 S.W.G. wire. Crane rails will have connections to the riser at every 100 ft.

Bare stranded copper conductors will be provided.

To avoid galvanic action only copper and copper alloy conductors will be used.

Layout of main leads is shown in fig. 3.

In the design of earthing mats considerable work had been done in Irrigation Department, Rihand Design Directorate and their designs were of great help to me in finalizing the problem. I gratefully acknowledge their help and of my other colleagues with whom I had a series of discussions on the calculation of potential gradients and their effect on safety of apparatus and workers.

HEATER-DRAIN CONTROL REQUIREMENTS

Design factors which contribute to tighter control requirements stem from the trend to higher capacity and wider-range operation. As unit sizes soar beyond the 200-mw rating, through flow in feedwater heaters becomes very high relative to storage capacity of level-control area. Maximum amount of feedwater is heated in minimum amount of power-plant space. This condition is particularly noticeable in designs where vertical heaters are used and in horizontal heaters where sub-coolers occupy a major portion of condensate space. Need to handle wide range of load swings during a 24-hour period is second factor forcing more exacting selection of heater-drain controls. This brings into play many factors not apparent in large-sized base-loaded units. Feedwater-heater systems usually incorporate various combinations of three basic drain-control systems common to all modern power stations: 1) Conventional cascade system, where high-pressure heater drains directly to lower pressure heater or condenser with level-control valve located in line between the two vessels. 2) Similar to system 1 except heater is drained by a condensate pump and level-control valve is between pump and lower pressure system. 3) Differs from system 2 in that heater drain pump discharges into relatively constant-pressure system such as suction side of boiler-feed pump or discharge of main condensate pumps.

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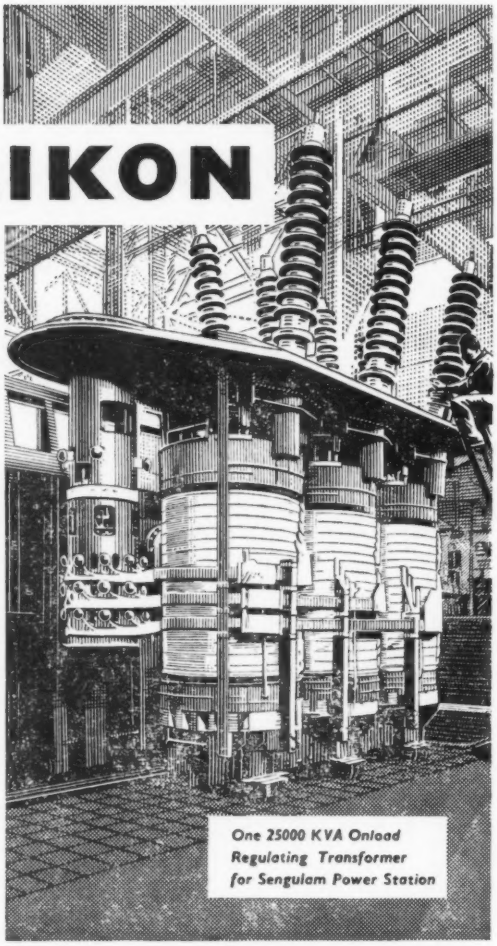

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INS/G/13

Fire Protection of Rihand Power Station

By O. N. SAXENA

THE various possible sources of fire and the extent of conflagration likely to occur in fire hazardous occupancies at Rihand Power Station have been carefully weighed and estimated in an effort to plan effective, efficient, adequate and reliable layout of fire fighting equipments.

The various occupancies, for which automatic fire extinguishing and detecting systems have been planned are as follows:—

A. *Generators* .. 5 Nos. each of 55,500 kVA capacity.

B. *Oil Occupancies*

(i) Transformers 5 Nos. each of 60,000 kVA capacity.
7 Nos. each of 500 kVA capacity.

(ii) Central Oil purifier for Transformer Oil and two oil tanks for the storage of dirty and clean oil.

(iii) Oil Storage Rooms.

The other occupancies of the Power Station which are less hazardous, will be protected by manually operated portable CO_2 cylinders.

A. FIRE PROTECTION OF GENERATORS

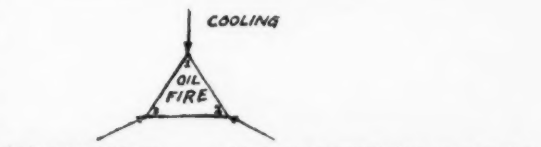
The apparent cause of 'Fire' in a Generator is attributed to the failure of insulation of the windings which may develop into electrical short circuits with consequent overheating and burning of the conductors and lead to 'Fire'. In such eventualities it is essential, that the machine must stop 'generating' and feeding into the fault as the source of fire must be removed before successful operation of extinguishing the fire can be achieved. The protective gear for Rihand Generators includes Generator Differential, Transverse Differential and Earth Fault Relays for guarding against the short circuits of the stator windings. These relays have been arranged for automatic shut down of the machine and in addition for the automatic operation of the fire fighting equipment. As the alternator is closed air circuit self ventilated, it is considered that release of CO_2 can be effectively used for promoting an incombustible or inert atmosphere inside the machine. For the satisfactory operation, it is essential that the concentration of CO_2 be sufficient so as to dilute the air to a point where combustion is not possible. This will be achieved by providing sufficient capacity of CO_2 and distributing it uniformly inside the Generator by nozzles and pipe work. A battery of CO_2 cylinders of adequate capacity so as to meet the requirement of one complete discharge for one machine will be installed at turbine floor and connected to the nozzle system of all the five units through weight operated selector valves controlled by electric release trips.

A second battery of CO_2 cylinders of the same capacity as the first, with interconnecting pipe work, valves and release trip will be installed as standby. The control circuits for CO_2 have been designed to carry out the tripping function of the selector valves either automatically by the generator protective devices referred above or manually by operating upon push button switch on the turbine control panel. When the electric release trip unit is energised, an auxiliary contact on the same initiates alarm which appears on the turbine control panel indicating the tripping function of the CO_2 release unit. The selector valves will be capable of local hand control under emergency conditions. The testing of the valve will be carried out by operating the test switch connected in parallel with the Push Button, after inserting the flagged test pin in the lug which will allow electric or manual operation of the trip mechanism without permitting the cylinder operating weights to be released. The discharge of CO_2 from the battery has been arranged to occur in two stages i.e., initial discharge and delayed discharge for efficient control of fire. Half of the cylinders will be connected to one set of nozzles for initial discharge and the remaining half to another set of nozzles through hooded horns for delayed discharge. Routine weighing of the CO_2 cylinders will also be carried out as a part of maintenance schedule.

B. FIRE PROTECTION OF OIL OCCUPANCIES

Principles of attacking Oil fires

The presence of oil—an inflammable liquid, in various occupancies listed above is a constant potential threat of fire and the effective use of water to meet this challenge, is well known. The application of science to this problem by various fire protection organisations of the world has revealed that oil fires can be attacked by water simultaneously with three important effects enumerated below.



THREE SIMULTANEOUS ATTACKS ON FIRE

1. *Cooling*

The dispersion of water into fine spray partially evaporates it and results in absorption of heat from the atmosphere thereby producing cooling effect. The ignition of oil takes place at a particular temperature which is defined as the "Fire point" and spontaneous ignition of oil vapour occurs at a comparatively lower temperature known as flash point. The fire and flash point for transformer oil are approximately 200°C and 160°C respectively and cooling of oil to a temperature lower than these renders it non-inflammable. The cooling action of spray produces a temperature much below flash point, thus arresting formation of oil vapours and development of explosive atmosphere.

2. *Dilution of Combustible Atmosphere*

The combustion of oil or any substance needs a definite concentration of active air "Oxygen". The formation of water vapours or steam dilutes the air and if this dilution is sufficient above the oil surface, the ignition of oil will be rendered difficult.

3. *Emulsification of Oil*

The property of oil to form emulsion with water when both the fluids are mechanically agitated opens the third channel for attacking the oil fires. A microscopic examination of this emulsion reveals that it comprises of tiny globules of oil each sorrounded by a film of water. This film of water, forms a fire proof covering for each globule of oil. The mechanical agitation of oil and water can be achieved by producing a forceful spray of water with sufficiently high velocity and momentum so as to promote a quick development of emulsion at the surface of the oil, thereby rendering it incapable of burning. This emulsion is of temporary character and oil and water separates out after the spray is turned off. The spray does not have any detrimental effect on the oil.

The advantages of this agent in its use on oil filled electrical equipments like transformers are not offset by its inadvertent application on live equipments as the spray consisting of fine broken water particles have a very high resistivity to the flow of current and also the flashover voltage never falls below that of the standard Rain test. The water spray would in fact wash away the conducting particles of the soot deposited on the bushings which may otherwise promote flashover.

GENERAL SCHEME OF PROTECTION

The scheme of protection broadly includes the following:—

1. Steps to reduce the chance of an outbreak.
2. Precautions to limit the area of the possible conflagration.
3. Arrangements for a reliable and adequate source of water supply, approx. at 90 P.S.I.
4. Arrangements for automatic detection of fire.
5. Arrangements for producing water spray in the fire zone and automatic operation of the same by fire detection system.
6. Arrangements for local and remote signal of fire and remote indication of defects in the system which may result in premature operation of the system.
7. Arrangements for the disposal of burning oil, water and smoke.
8. Arrangements for proper maintenance and periodic testing of fire fighting equipments.

A water tank of 20,000 gallons capacity will be provided on the top of the Elevator tower in Block No. 34 of the Dam. The base of this tank will be at an elevation of 924′ i.e., 241′ above the transformer floor and will be fed by electrically operated pumps. This tank will not be used for any other purpose except that of fire. A 8″ water main from this tank will run longitudinally along the Power House Transformer Bay and will feed to nozzle system of each section of the Bay. A main control valve will be installed on this pipe at a point where this enters into the Bay. This 8″ dia water main will be connected to a 6″ dia pipe at the end of its run on the Transformer Bay and the latter will feed the nozzle system of oil purifier, oil tanks and oil storage rooms at elevation of 650 through a Sectional Control Valve.

The signal of fire and defect of the fire protection system will be communicated to the Fire Alarm panel suitably located in the Control room. This panel comprising of 18 sections equipped with electric Siren, test switch and audible alarm isolating switch. In the event of fire the common inscription 'Fire' and the inscription indicating the zone of fire will be simultaneously illuminated and the siren will also be sounded. In the event of low air pressure in the detector system of a particular zone, the corresponding inscription will be illuminated and the siren will be sounded.

FIRE PROTECTION OF TRANSFORMERS

The steps for reducing the probabilities of fire in transformer include installation of various protective relays so as to initiate shut down of the unit under unhealthy conditions. The precautions for limiting the area of possible conflagration will include the construction of fire walls so as to generally house one 60,000 kVA and one 500 kVA transformer of each unit in one compartment and thus reduce the possibility of open communication of fire from one section to another. The burning oil and water will drain into a common gutter of sufficient capacity running along the transformer bay but at a considerable distance from the transformers.

A group of water projectors (open nozzles) suitably positioned and connected to a network of piping system around the transformers of each section or unit, will be used for producing a cone of discharge

of water in the form of spray. The arrangement of nozzles, the velocity of spray, range and spray angle will be such as to command the entire area of conflagration and produce high extinguishing efficiency. The discharge characteristic of the nozzles will be such as to discharge 18 g.p.m. of water at 50 P.S.I. and cover an area of 25 sq. ft from a height of 20'. The whole of the piping system will be so graded as to produce uniform discharge from all the nozzles and will be supported on steel structures which will be effectively earthed. The system of water projectors around 60,000 kVA transformers will be connected to the system of water projectors around the 500 kVA unit transformers and supply of water to both the systems will be controlled by a single valve assembly known as M.F. valve.

Thus in the event of fire on either of these transformers all the nozzles of each system will simultaneously come into action, thereby deluging the entire body of each transformer and the immediate floor area with high velocity forceful spray of water. The inlet of M.F. valve will be coupled to 8" water mains and the outlet from this valve will be connected to the system of water projectors. One stop valve each on inlet and outlet connections will be installed and each of these will normally be locked in open position by a strap. A third connection from the M.F valve will be made to a detector pipe work which will be charged with air under 40 P.S.I pressure and will carry sprinkler detectors suitably positioned in the fire zone. The M.F valve will normally be held in the closed position by the pressure of air. The sprinkler detector which will be held normally in the closed position by a quartzoid bulb will open and relieve the air pressure from the detector pipe work and M.F valve on the occurrence of fire since quartzoid bulb is filled with volatile liquid which vapourises and breaks the quartzoid bulb when a fixed temperature is reached. The operation of any of the sprinkler detectors will relieve the air pressure from the M.F valve and cause it to open at 10 p.s.i air pressure and feed water to the projectors. The manual operation of M.F valve will also be possible and could be achieved by operating a valve in the detector system so as to relieve the air pressure. The flow of water through the M.F valve will operate an electrical alarm unit and water motor gong for remote and local indication of fire. Loss of air pressure to 25 P.S.I in the detector system caused by leakages, will operate Diaphragm Alarm switch in the M.F. valve which will communicate a warning signal to the Fire Alarm Panel before premature operation of the valve takes place. A pressure guage each on the water inlet and detector connections will be installed so as to facilitate maintenance. A test valve on detector pipe work shall be used for periodic testing purposes. The supply of compressed air to the detector system will be obtained from a common bus for the transformer bay through a non-return valve and stop valve.

FIRE PROTECTION OF OIL STORAGE ROOMS

These rooms are meant for housing lubricating oil drums, sundry oil store, grease, portable oil purifier, oil testing equipments, central oil purifier and oil tanks etc. Partition walls or fire walls between each of the occupancies mentioned above will be erected so as to limit the area of conflagration. Fire doors between the power house and the oil rooms will be installed and these will be held in open position by fusible elements which will melt in the event of fire, thereby resulting in automatic closure of the doors. The provision of chilling drains for the disposal of burning oil and ventilation duct system for exhausting smoke from these rooms, have been made. The probabilities of fire in this being less compared with the transformer deck a comparatively cheaper and simpler scheme for protecting these occupancies has been envisaged. This is quite similar to the scheme described earlier under transformer. The difference lies only in the method of controlling the application of water under conditions of fire. The detection of fire and control of water to the projectors in central oil purifier and oil tanks will be carried out by a valve known as "Mulsifire control". This valve will be connected to a small group of water projectors. The principle of operation of this valve is similar to that of sprinkler detector. The rupture of quartzoid bulb on the valve in the event of Fire releases water to the group of projectors or nozzles. A number of such control valves along with associated projectors with them will be suitably distributed around the area. The functions of detection of fire and control of water for other rooms will be carried out by number of sprinklers suitably located in these rooms and each sprinkler will individually function both as detector and projector. The water supply to the Mulsifire controls and sprinklers will be obtained from a common main running in these rooms. In the event of operation of any one of the Mulsifire control valves or sprinklers the flow of water through the sectional control valve will operate Alarm Valve and Water motor gong for remote and local indication of fire.

The total expenditure on the fire protection scheme described above will be approximately Rs. 2.16 lacs and will protect property worth about Rs. 150 lacs. The interest and depreciation at 6% on Rs. 2.16 lacs amounting to Rs. 13,000/- per annum may be looked upon as an insurance premium on the property worth Rs. 150 lacs against fire.

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SISTA'S-B-144

Design Criteria for Lighting of Rihand Dam and Power Station

By SANTOKH SINGH

THE lighting of modern dam and power station presents problems both technical and psychological. With the realisation of visual comfort in increasing efficiency, the psychological aspect has assumed more and more importance than the technical. It is now generally recognized that good lighting is essential to careful operation and sound maintenance. The following points need consideration for arriving at such a scheme:

1. Level of illumination.
2. Type of source of lighting.
3. Type of fittings.
4. Any local requirement or special circumstances demanding consideration or attention.

LEVEL OF ILLUMINATION

The level of illumination depends upon the purpose for which the lighting is required i.e., upon the fineness of work, degree of contrast or the nature of the objects on which work is to be done. The finer the work, and lesser the contrast, higher is the level required. As high level increases the cost of installation, a thorough study is needed in selecting the same. Table No. I gives levels of illumination which have been selected for Rihand Dam and Power Station.

TABLE I

Serial No.	*Location*	*Levels adopted Lumens/Sq.ft.*
1.	Intake unit control chambers and hoist m/c platform	4.0
2.	Galleries and adits	1.5
3.	Lobbies	8.0
4.	Sluice gate chambers	4.0
5.	Battery Room	7.0
6.	Control Room	20.0
7.	Machine Shop	10.0
8.	Offices	10.0
9.	Cable Gallery	3.0
10.	Generator floor	8.0
11.	Turbine floor	7.0
12.	Switch gear room	10.0
13.	Retiring room	5.0

TYPE OF SOURCE OF LIGHTING

There are four most common sources of lighting.

(i) Incandescent lamps.
(ii) Mercury Vapour lamps.
(iii) Sodium vapour lamps.
(iv) Fluorescent lamps.

In assessing the relative merits of each of the available types, factors such as efficiency, rated life, brightness, colour rendering property, initial cost, and replacement cost need special consideration. Given below is a comparison of the general performance of several sources, now available, in the form of a table.

TABLE II

Serial No.	*Type of source*	*Efficiency Lumens watts*	*Life in hrs.*	*Brightness Cd/Cm²*	*Colour*	*Initial cost*	*Replacement cost*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1.	Incandescent lamp ..	15	1000	700	Accepted Warm White	Low	Low
2.	Mercury vapour lamp with fluorescent coating ..	38	4000	23	White light	High	High
3.	Sodium vapour lamp. ..	63	4000	10	Mono Chromatic (yellow)	High	High
4.	Fluorescent lamps. ..	40	7500	0.5	Available in daylight, white light, Delux etc.	High	Relatively Low.

In addition to the above, aesthetic effect of lighting and local conditions such as air conditioning of building etc., play important part in deciding the source. Because of white colour and glare free light, fluorescent tubes are finding greater use, particularly in lighting of offices and public places. Places which are air conditioned are also generally lighted with fluorescent tubes as these add less heat load due to their efficiency being high. As mercury vapour lamps and sodium vapour lamps are imported material, their use is kept to a minimum.

After giving due consideration to the above and connected factors, the type of lighting sources selected for the various places at the Rihand Dam and Power Station are as follows:

TABLE III

Serial No.	*Location*	*Requirements*	*Type of light source selected*	*Type of lighting fittings*
(1)	(2)	(3)	(4)	(5)
1.	Control room.	High level (20 lu/sq. ft.) of glare free lighting, colour discrimination essential, room to be air-conditioned, aesthetic effect and peaceful atmosphere desirable.	Fluorescent lamps.	Decorative fittings suitable for recessed mounting in the false ceiling.
2.	Power House (Main floor)	Not very high level of illumination (8 lu/sq. ft), glare not of much importance as lamps are to be fitted in the ceiling (56′ high), colour discrimination not essential but desirable, the lighting fittings to be suitable for high bays.	Mercury vapour + Incandescent lamps.	Trough type fittings suitable to accommodate combined mercury vapour lamps & incandescent lamps.
3.	Office Rooms & lobbies in dam.	Comparatively high level of illumination (10 lu/sq. ft.), light should be glare free and colour discrimination essential, office rooms are being airconditioned, aesthetic effect desirable.	Fluorescent lamps.	Decorative fittings having louvers etc. for diffusion.
4.	Turbine Floor	Not very high level of illumination, colour discrimination desirable, ceiling only 12′ high from floor level, aesthetic effect not essential but desirable. Glare free light desirable.	Fluorescent lamps.	Trough type fluorescent fittings.
5.	Cable galleries	Low level of glare free lighting, ceiling at low height from floor level, the objects have very low reflection factors.	Incandescent lamps	Prismatic bulk head fittings
6.	Galleries	Very low level of illumination is required, ceiling very low, sides have very low reflection factor. No aesthetic effect is required.	Incandescent lamps	*For galleries of Power House*:— Prismatic bulk head fittings. *For galleries of Dam*:— Recesses have been provided to accommodate lamps and to be used as reflectors.
7.	Intake unit control chambers, hoist m/c platform & sluice gate chambers.	Low level of illumination is required. Sides have very low reflection factors. Aesthetic effect and colour discrimination not required.	Incandescent lamps.	Industrial type reflectors.

TYPE OF FITTINGS

Suitable type of fittings have been selected depending upon the nature and use of each place and have been given in the last column of above table No. III.

In addition to the above, the lighting of roads, switchyard and spillways etc. presents their own problems and in short the following schemes have been proposed:

Road Lighting

The roads on the dam top and the approach to the power station will be 31′ and 50′ wide respectively including walk ways along both sides. Post top, 4×20 W fluorescent fittings have been selected for the lighting of the above roads. On the dam road there will be a single row of these lights spaced 80′ along the down stream edge of the road and with a height

(Continued on page 118)

Rihand Project: Planning and Progress

By S. K. SHOME

RIHAND Project envisages to utilise the water potential of Rihand river in district Mirzapur for development of hydro power. A 304 feet high cement concrete dam would store 8.6 million acre feet of water and a power station situated at the toe of the dam will use this storage to generate power. The power house will have an ultimate capacity of 3,00,000 kW but initially only five units of 50,000 kW each will be installed.

History of development of Rihand Project dates back to 1944 when the topographical, hydrological and geological surveys and investigations were started. A skeleton project was framed in 1948 on the basis of these surveys, but final shape to the project was given in 1954 after comprehensive studies and investigations. During all this period, the project remained under investigation stage and full scale construction work could only be started in April 1954, when the financial arrangements for funds required for the execution of the project were finalised with the Government of India.

With the decision to take up the project all the preliminary prerequisites for the construction of the main dam and power house were taken up in phases. First came the road link and the bridge over river Sone. Then came the colony for the government staff, hospital, school and other utilities. To meet the requirement of the construction power which was estimated to have a peak of 6000 kW, a 3000 kW diesel station was set up at the Dam site and this was later supplemented by 6000 kW thermal plant at Chopan.

While the construction of road, bridge and colony was in progress, specifications and other tender documents for main civil engineering works were prepared and tenders invited. Global tenders were received in December 1954 and the work was awarded to M/s. Hindustan Construction Co., Ltd., of Bombay in March 1955.

The contract calls for completion of all the works in the dam, power house and appurtenances by June 30, 1961. Initial planning of the job with a view to select, procure and install the construction plant was done and January 1957 was set as target for start of concreting operations.

With the available time for completion the average rate of concreting per day worked out to 80,000 cft and taking the peak capacity as 150% of the average rate the entire construction plant was selected for concreting at a peak rate of 1,20,000 cft per day (Maximum concrete poured on any date uptil now has been 1,24,713 cft.). Timely procurement and installation of construction equipment required most careful planning specially in the face of ever increasing import difficulties and transport bottlenecks. Selection of equipment and orders for procurement were, however, finalised by October 1955. The construction of foundations for various construction machinery was also commenced in advance so as to be ready by the time the machinery arrived.

The construction equipment started arriving in October 1956 by which time the road link from Robertsganj and the bridge across Sone were ready, for the transport of the heavy equipment to the site of works.

The construction equipment mainly comprised of:

(i) Two 20 ton cableways over a span of about 3000 ft. (longest ever used in the world).
(ii) Two 10 ton cableways over a span of about 2100 ft.
(iii) Batching plant with four mixers of 4 cu.yards each.
(iv) Refrigeration plant of 990 tons capacity.
(v) Two ropeways of 200 tons per hour capacity and 11700 ft. long.
(vi) Primary and secondary crushing units for aggregate production at 400 tons per hour.
(vii) Quarrying equipment including shovels and dumpers for haulage of quarry stone to the crushers.
(viii) A miscellany of allied equipment such as belt conveyors, welding sets, pumps, vibrators, compressors etc.

The 10 ton cableways, one rope way, one primary crusher of 150 ton per hour capacity and two units of refrigeration plant were available with the contractors from their previous job on Vaiterna Dam works. This equipment was installed as and when it arrived at the dam site.

A detailed schedule of construction was drawn up in the meantime and the 592 lacs cft of concrete required for completion of dam was proposed to be poured in the different seasons as below:—

I	Season	Jan.	1957	to	Sept.	1957- 34	lacs cft.
II	„	Oct.	1957	to	„	1958-191	„ „
III	„	„	1958	to	„	1959-176	„ „
IV	„	„	1959	to	„	1960-146	„ „
V	„	„	1960	to	June	1961- 45	„ „

Diversion of the river was carefully planned. The work area was planned to be isolated during normal times by constructing upstream and downstream masonry coffer dams and confining the flow of the river to a diversion channel crossing the dam align-

ment on Blocks 25 and 26. These works were completed by March 1957.

To ensure uninterrupted working in the power house during monsoons it was proposed to isolate this area by building a coffer dam allround above the highest flood level. This work was scheduled for completion before monsoon of 1958.

An immediate start was made on the excavation of foundation and by March 1957 most of the excavation was completed and only surface preparation of blocks prior to concreting remained to be done.

A minor setback in the procurement of plant was caused at this stage due to Suez Canal crisis and the arrival of certain construction equipment was delayed. Some components of batching plant itself arrived as late as March 1957 and the concreting operations could, therefore, commence on April 5, 1957 with the help of two 10 ton cable ways as the 20 ton cableways were yet to be installed.

Concreting during the first season (April '57 to August '57) was confined only to the raising of these blocks of the dam over which the tail tracks of the cableways were to be installed. Concrete was brought from the batching plant by the 10 ton cableways and lowered on the downstream side of the blocks. Stiff-leg cranes, erected temporarily for concreting this season, picked up these buckets and transported them to the blocks for placement. Only 11 lacs cft of concrete could be placed in the season.

The tail track of the cableways were put through in September '57 and the concreting of the second season was commenced in October 1957 still with the smaller 10 ton cableways.

Erection of the two 20 ton cableways was continued simultaneously and these were brought into commission in November 1957 and January 1958 respectively. Thus full scale concreting operation could start only in March 1958 when all the four cableways became operative.

There were innumerable teething troubles with the construction plant specially at quarry and the crushing plant due to heavy wear on account of the hard and abrasive nature of stone.

The 20 ton cableways being the longest in the world to be ever used, also required maximum care and caution in handling and could not, therefore, be geared to full scale efficiency straight away.

Spillway blocks were kept low and the right abutment and the intake blocks were raised well above the high flood level.

In order to maintain continuity of work in the power house, the work area in the power house pit had to be isolated from monsoon flood by the construction of a 70 ft high masonry coffer dam comprising 8,00,000 cft. of masonry. The work on this coffer dam was started in February 1958 and completed in June 1958.

A conduit was left in one of the spillway blocks and the river was diverted through this conduit by damming the upstream of the old diversion block. This new diversion was effected in April '58 and the full heat was then turned on the concreting of old diversion blocks. In a race against time these blocks were prepared and concreted to levels above H.F.L. before the monsoon broke. The new diversion conduit is provided with a gate on the upstream face of the dam so that it can be sealed off by lowering the gate and then plugging the conduit with prepakt concrete.

In the second season only 152 lacs cft, of concrete could be placed and the progress of concreting at this stage was 62 lacs cft. behind the original scheduled target.

The plant was however in top condition. The quarry had stabilised and the 20 ton cableways had really started delivering the goods. With all these points in favour a higher revised target of 239 lacs cft of concrete was set for the third season and a detailed monthwise and blockwise programme was drawn up. Shuttering and form work requirements were worked out in detail and arranged for. Staff on concreting and different preconcreting operations were supplemented and all other steps were taken to meet the higher requirements of an increased work load. Work for third season was commenced in October 1958. Wonderful progress was maintained throughout the season and 249 lacs cft of concrete was placed upto end of season bringing the upto date total to 412 lacs cft against a target of 401 lacs cft.

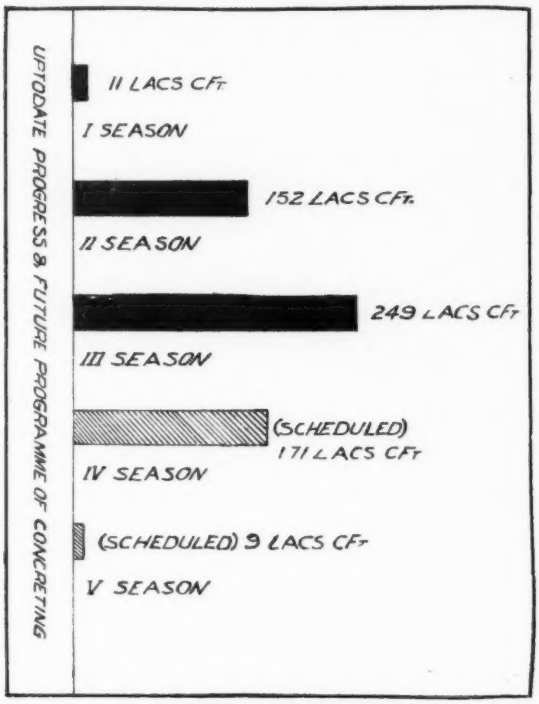
Thus with careful planning and co-ordination the job which was way behind schedule was pulled ahead of schedule. During this season concentration was entirely on the right abutment and the intake blocks. Right abutment was pushed all along so as to ensure the completion of the blocks to top from right end of the dam to the intake by June 1960. This will enable us to lay the track for the intake gantry crane upto the intake block so that as soon as the crane arrives, it could be erected and moved over to the intake blocks for the erection and installation of the penstock gates. All the buckets and both the retaining walls were completed so as to finish all work in the river bed.

Drilling and grouting of the intake blocks from the foundation gallery was started and three stages of 25 feet each were drilled and grouted in the intake section. This work shall continue now as the blocks keep rising up (Specifications demand construction of a block to a ht. of 100 feet or to top before grouting is started). Foreign exchange difficulties are delaying the procurement of bits but it is still expected that all drilling and grouting will be completed before the dam fills up to full supply level.

The programme for the fourth season has been finalised after detailed studies. A target of 171 lacs cft has been set up for this season and the remaining concrete is to be placed after the fourth season.

It is proposed to raise the spillway to the crest level in this season and complete to their full height the entire right abutment and the intake blocks. The

left abutment will be raised to sufficient height above spillway so that a normal flood of 4.25 lacs cusecs could be routed through the spillway, (assuming that the reservoir is full upto spillway crest level before the flood) without the water overtopping the left abutment blocks.



PENSTOCK PIPES

Water from the reservoir to the turbines will be carried through 16 ft. diameter steel penstock pipes embedded in concrete in the intake dam. Fabrication of these pipes comprising about 1000 tons of steel was carried out at site. Plates for fabrication of penstock pipes could not be procured in time and the intake dam had therefore to be built in two stages. It was obligatory to build the intake dam upto above high water level before June 1958 to avoid flooding of the power house pit and to enable the work on construction of power house to continue during the monsoon. The intake dam was, therefore, left in steps. As soon as the plates were available the work of fabrication of pipes was pushed ahead round the clock and by end of the season all the pipes were manufactured. The pipes were duly tested and installed and the blockouts were completed in all respects by end of third season. In this way the problem of delay in procurement of steel was successfully encountered and the construction was so phased that by end of third season the work was on schedule.

The penstock pipes were manufactured from steel plates of thickness varying from $\frac{1}{2}$" to 1" and were of welded construction. The longitudinal joints of the pipe shells were radiographed to ascertain the quality of welding. Hydrostatic pressure testing of the pipes was also done to ensure the quality of material in the plates and of welds in the girth joints. Most of the welding was done by automatic welding machines and in case of manual welding, the welders who were employed on the job had to pass tests in different welding positions before they were allowed to work on the penstocks.

POWER HOUSE AND GENERATING PLANT

The power house excavation was completed by May 1957 and the rock covering lifts were laid in the first season. Next year the work area had to be desilted. The units were again made ready for concreting by December 1957.

Draft tube liners in all the bays were embedded in the scond season and a part of the first stage concrete was placed.

Full concentration of efforts was made on the power house during the third season and by April 1959 the erection bay and the first unit bay were constructed to a stage to start turbine erection. The fabrication of steel columns, girders and roof trusses for the power house superstructure was started concurrently. The gantry cranes of the power house were installed, tested and commissioned. By end of third season all the first stage concrete was laid and most of the steel superstructure was up. Scroll case assembly was complete in one unit bay and was in progress in other two units.

The second stage concrete has now been started. The second stage concrete will be poured in 2.5′ lifts with an interval of 2 to 3 days between successive lifts. According to the planning the entire second stage concrete of one unit bay is to take 4 months but the progress at site uptil now has been better than that.

The assembly of the first rotor has started in the erection bay. It is proposed to carryout the test of the first machine during the months of April, May 1961 and the remaining four machines by December 1961.

PERMANENT EQUIPMENT

The different equipment required for Rihand Dam are listed below:—

1. Trashracks.
2. Intake stoplogs.
3. Draft tube gates.
4. Bulkhead gates for sluices.
5. Penstock gates and hoists.
6. Passenger Elevator.
7. Power House Elevator.
8. Freight Elevator.
9. Slide gates and Hoists.
10. Tainter Gates.
11. Intake Gantry Crane.
12. Draft tube Gantry Crane.
13. Bulkhead for penstocks.
14. Expansion Joints.

The first four items are being manufactured at the

departmental workshop at Bareilly and Roorkee. Though there was quite a lot of difficulty in procurement of steel, yet it is certain that all equipment will be received at site in due time. Trash racks and intake stoplogs are ready and part requirement of drafttube gates have already been fabricated. The embedded parts were planned and procured in concurrence with the concreting programme and only block-outs for guides have been left.

The Bulkhead have been manufactured at site while the expansion joints are being imported.

Items at serial 5 to 12 are being procured from open market by global tenders. It was initially apprehended that considerable time will be required for import of these equipment and as such preparation of specification were started by the Equipment Division as early as February '55. Specifications were completed equipmentwise and global tenders called for each equipment separately. The prevalent practice of equipment procurement from invitation of tenders to the award of the contract is as follows:

1. Tenders are invited, scrutinised and the recommendations are made by the Chief Engineer to the Rihand Control Board.

2. Rihand Control Board transmit the papers to the C.W.P.C. after giving their recommendations.

3. C.W.P.C. carry out a full technical scrutinity of the proposals and then submit their recommendation to the Government of India, Irrigation Ministry.

4. Irrigation Ministry further scrutinises the tenders and sends same to the Development wing for clearance in respect of items requiring foreign exchange. After clearance from the Development Wing, the final proposal goes to the Finance Ministry for release of foreign exchange.

5. After foreign exchange is released, the project authorities complete the formalities of awarding the contract and obtaining the import licence.

6. Delivery period of the tender counts from the date of award of contract.

This entire process is very time consuming and even with such an early start, the project is facing some difficult situations regarding procurement and installation of permanent equipment. All this clearly indicate that for all future jobs the planning for procurement of equipment should be taken up very early and should be pursued on a top priority basis right from the beginning.

(Continued from page 114)

DESIGN CRITERIA FOR LIGHTING OF RIHAND DAM AND POWER STATION

of 13'—15' above road level. This arrangement is estimated to provide 3800 lumens per 100' of the road. On the approach road to the Power Station there will be two rows of lights, one along each side of the road. The lights will be similar to the above but will be staggered and spaced 100' apart. This arrangement is estimated to provide 6000 lumens per 100' of road.

Switchyard Lighting

For this purpose, the installed capacity of the flood light projectors has been arrived at by taking 2.5 lumens/sq.ft. of switchyard area. The projectors have been spaced all around the periphery of the substation. No light fittings have been installed inside the area to avoid insect trouble in rainy season. Also the projectors have been installed at such height that maintenance is easy without danger of proximity to live conductors etc. The projectors are further capable of being adjusted both in horizontal and vertical planes so that concentrated lighting may be obtained for any repair work in the switchyard.

Spillway Flood Lighting

Angle type flood light fittings with 1000 watts incandescent lamps will be located on the down-stream edge of dam top road in the spill-way sections for flood lighting the over-flowing water. The main and auxiliary reflectors of the fittings have been so selected as to give uniform light distribution over the entire surface of the falling water. The fittings will be glare-free from convenient observation positions.

Powerhouse Flood Lighting

Due provisions of flood lighting the down-stream face of the power house have been made. For this purpose 4-1000 W. flood light projectors, two on either side of tail race will be located 100' away from the down-stream face of the powerhouse.

Emergency Lighting

For this purpose a reduced scale of lighting has been accepted. Light fittings have been provided at all important points of operations, access passages and stairways. The control room has been amply lighted for this purpose. No such lighting has been provided for offices and in the dam, as no danger to human life or machinery is expected at such places, in the event of total supply failures. The emergency lights are arranged to switch-on automatically.

The cost of lighting of Rihand Dam is estimated to be Rs. 1.34 lacs and that of power station, switchyard and control building Rs. 1.71 lacs.

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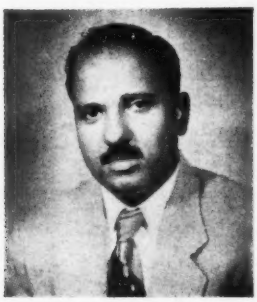

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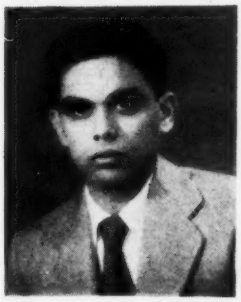

With his experience of designing and planning Quarry plant layouts, he is an asset to the ever-expanding activities of the company in the construction field.

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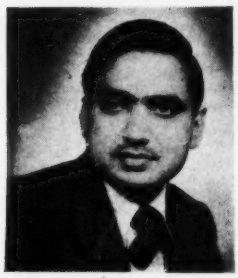

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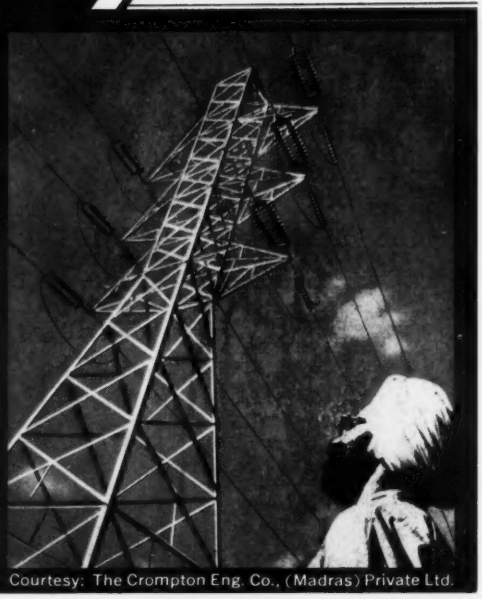

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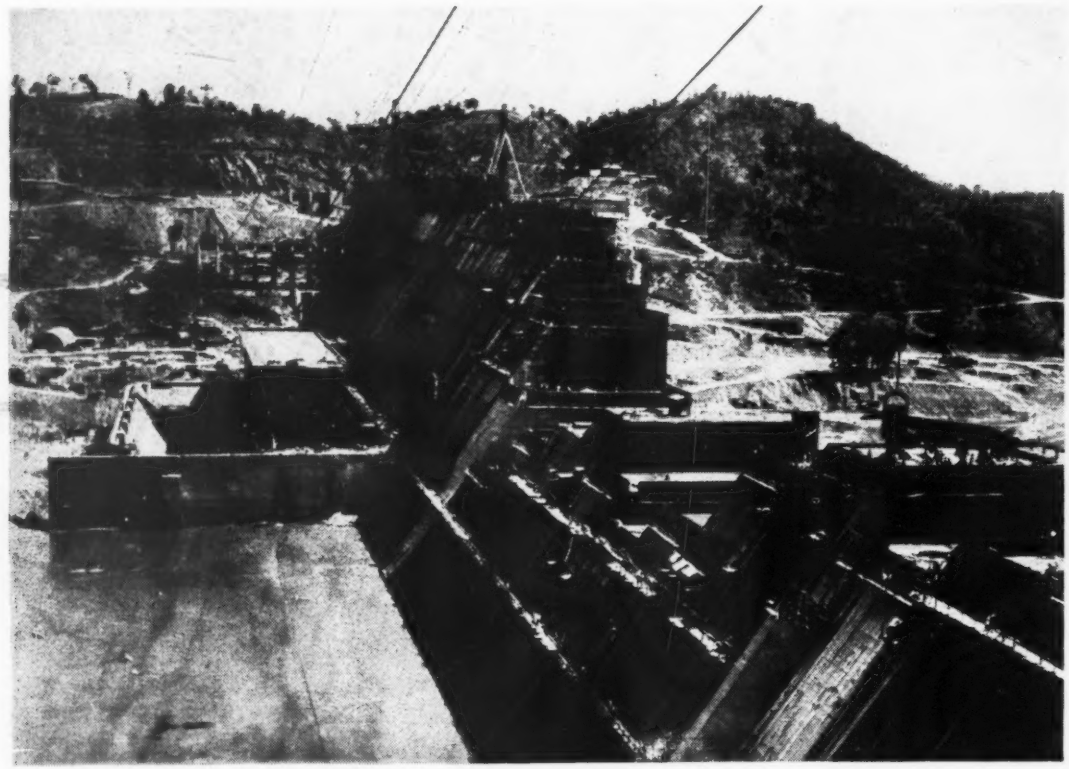

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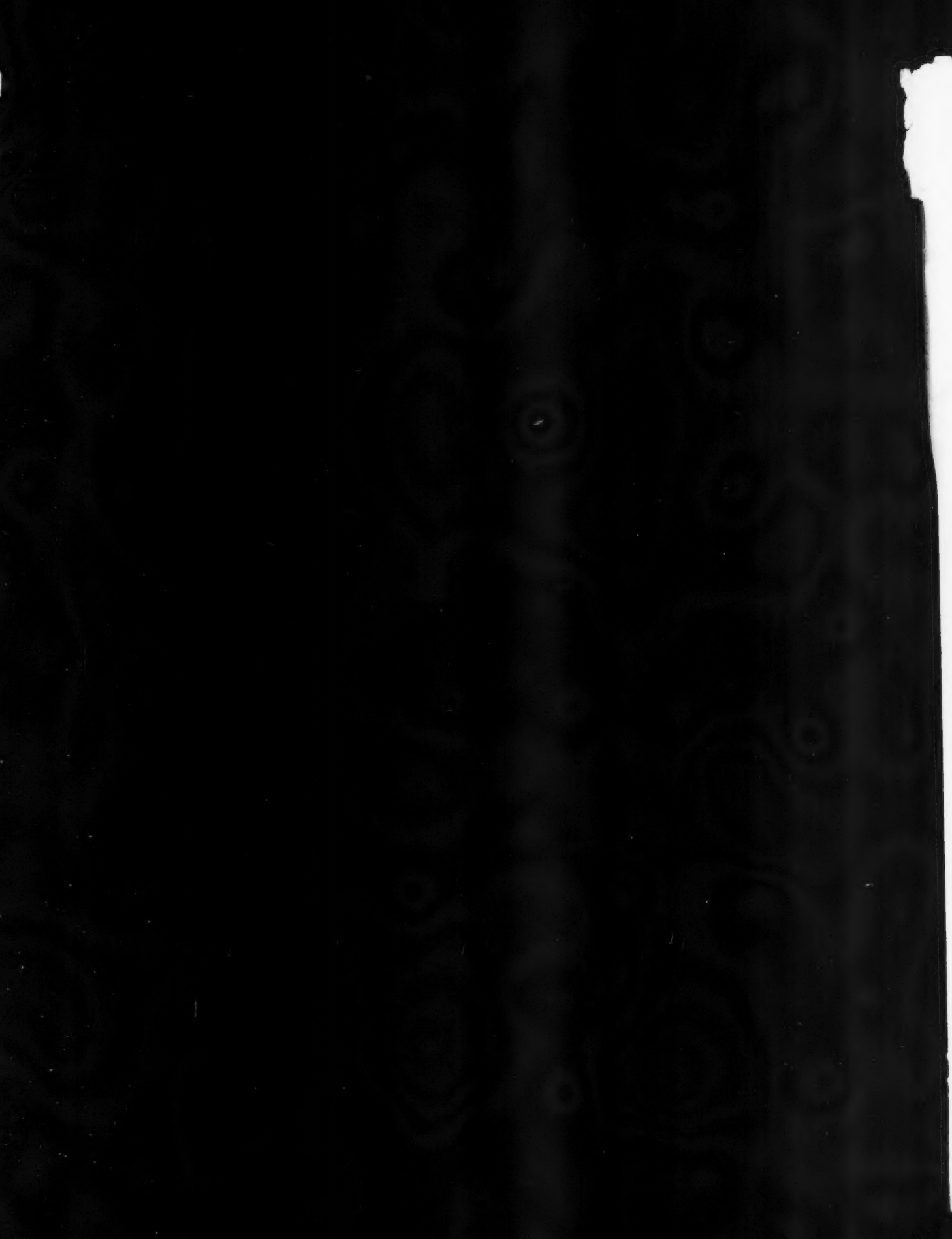
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